Geotechnics Incorporated

GEOTECHNICAL INVESTIGATION
GEOTHERMAL POWER PLANT
SALTON SEA UNIT NO. 6
CALIPATRIA, CALIFORNIA

prepared for

CalEnergy Operating Company, Inc.
7030 Gentry Road
Calipatria, California 92233

by

GEOTECHNICS INCORPORATED
Project No. 0673-002-00
Document No. 02-0022

February 5, 2002

Geotechnics Incorporated

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CalEnergy Operating Company, Inc.
7030 Gentry Road
Calipatria, California 92233

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Attention: Mr. Bernard Raemy

SUBJECT: REPORT OF GEOTECHNICAL INVESTIGATION
Geothermal Power Plant, Salton Sea Unit No. 6
Calipatria, California

Dear Mr. Raemy:

In accordance with your request, we have completed a geotechnical investigation of the proposed Salton Sea Unit No. 6 Geothermal Power Plant in Calipatria, California. The following report presents the findings, conclusions, and recommendations of our investigation. In general, our findings indicate that the site is underlain by relatively soft, saturated sedimentary deposits. These soils may be susceptible to liquefaction, and will likely experience substantial settlement as a result of the proposed foundation loads. In order to reduce the ultimate ground settlement to within the specified tolerances, the use of remedial grading, ground improvement and deep foundations will be necessary. Specific conclusions regarding the site conditions, and recommendations for earthwork, ground improvement and foundation construction are presented in the attached report. Where possible, we have included alternative remedial measures in our recommendations.

We appreciate this opportunity to provide professional services. If you have any questions or comments regarding this report or the services provided, please do not hesitate to contact us.

GEOTECHNICS INCORPORATED

Robert A. Torres, P.E. 43077
Principal Engineer

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1.0 INTRODUCTION

In accordance with your request, we have conducted a geotechnical investigation of the proposed Salton Sea Unit No. 6 geothermal power plant in Calipatria, California. This report presents the results of our subsurface investigation and analysis, as well as conclusions regarding the feasibility of the development, and recommendations regarding geotechnical aspects of the proposed improvements. This investigation and the associated geotechnical services were conducted in general accordance with the provisions of our Proposal No. 1-226, dated August 6, 2001.

2.0 PURPOSE AND SCOPE OF WORK

The purpose of our investigation was to evaluate the pertinent geotechnical conditions at the site, and based on these conditions, provide recommendations regarding the geotechnical aspects of construction of the proposed improvements. The recommendations contained herein are based on a surface reconnaissance, field exploration, laboratory testing, engineering analysis, and professional experience in the general site area. Design values may include presumptive parameters based on professional judgement. Our scope of work was limited to:

2.1 Review of available literature and published geologic maps pertaining to the general geotechnical conditions at the site. A list of relevant references is presented in Appendix A.

2.2 A field investigation program including borings, CPT soundings, percolation tests and a resistivity survey. The subsurface exploration of the site included 2 borings with a truck mounted hollow stem drill rig, and 9 soundings with a cone penetrometer (CPT). Pore pressure dissipation tests and shear wave velocity measurements were conducted at selected intervals on several of the CPT soundings. A variety of soil samples were collected from the borings for laboratory analysis. The CPT soundings, pore pressure dissipations, shear wave velocity measurements, and borings logs are presented in Appendix B. Two percolation tests were conducted at the proposed leach field location. The percolation characteristics are described in Section 5.1.3. A Wenner four point resistivity survey was conducted by M. J. Schiff and Associates at the location of the proposed grounding sub-station. The resistivity survey results are discussed in Section 5.1.4.

2.3 Laboratory testing of selected samples collected during the subsurface exploration. Testing included gradation, hydrometer, Atterberg, unit weight, moisture content, soil chemistry, expansion, direct shear, consolidation and R-Value. The laboratory test results are presented in Appendix C.

2.4 Assessment of general seismic conditions and geologic hazards affecting the area, and their likely impact on the project. Seismic analysis included the use of several commercially available computer codes, including EQFAULT, EQSEARCH and FRISKSP. The results from these analyses are presented in Appendix D.

2.5 Estimation of the liquefaction potential at the site in general accordance with the most recent improvements to the simplified method (Youd et al, 2001). The results from these analyses are presented in Appendix E.

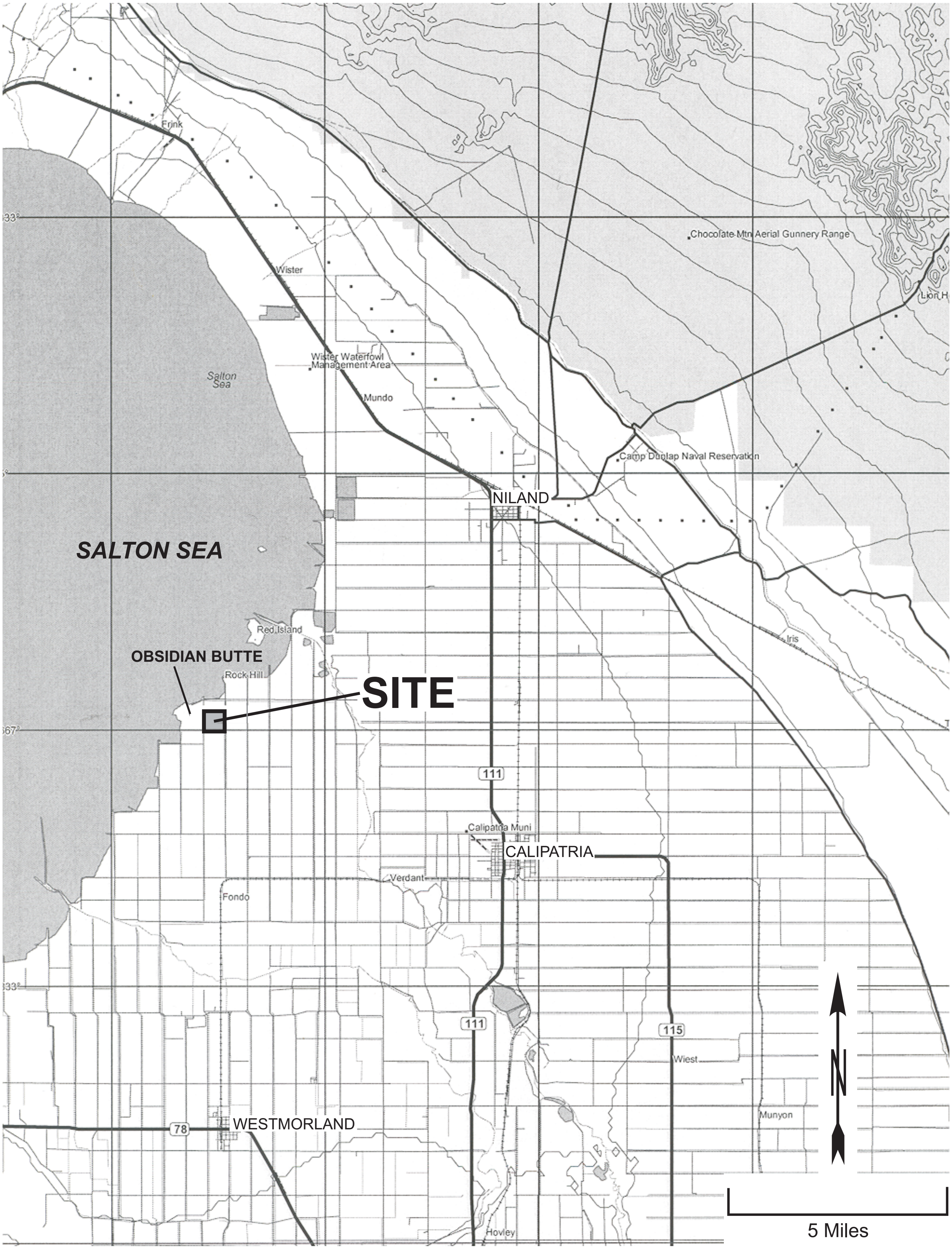
2.6 Engineering analysis of the field and laboratory data in order to develop recommendations for site preparation and remediation, earthwork construction, foundation and slab design, earth retaining structures, pavements and related improvements.

2.7 Preparation of this report summarizing our findings, conclusions and recommendations.

3.0 SITE DESCRIPTION

The subject site is located south of the Sonny Bono National Wildlife Refuge near the southern end of the Salton Sea in Imperial County, California. The site is situated southwest of the intersection of Gentry Road and the unpaved McNerny Road, as shown on the Site Location Map, Figure 1a, and the Detail Site Location Map, Figure 1b. Site access is provided by unpaved roads along the edges of the property. The western edge of the site is formed by an earthen dike and drain (Vail Lateral 5 Drain), which carries irrigation water to the Salton Sea (the drain water surface elevation is minus 224 feet). An earth embankment along McNerny Road borders the northern edge of the site. The embankments are about 7 to 10 feet high. The areas east and south of the site are farmland.

Site improvements consist mainly of earthen irrigation channels, valves and pumps for dewatering the field and, likely, drainage tiles beneath the crop. According to the aerial survey provided by Bibb Engineering, the farmland slopes very gently down from the southeast to the northwest. Based on the survey, surface elevations in the southeastern portion of the site are approximately 227 feet below sea level, whereas the northwest portions of the site are roughly at an elevation of 232 feet below sea level. It is our understanding that the Salton Sea level is approximately at an elevation of 227 below sea level. The approximate layout of the site is shown on the Site Plan, Figure 2.



Reference: Delorme 3-D Topo Quads, 2002.

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SITE LOCATION MAP

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FIGURE 1a

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Reference: Delorme 3-D Topo Quads, 2002.

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DETAIL SITE LOCATION MAP

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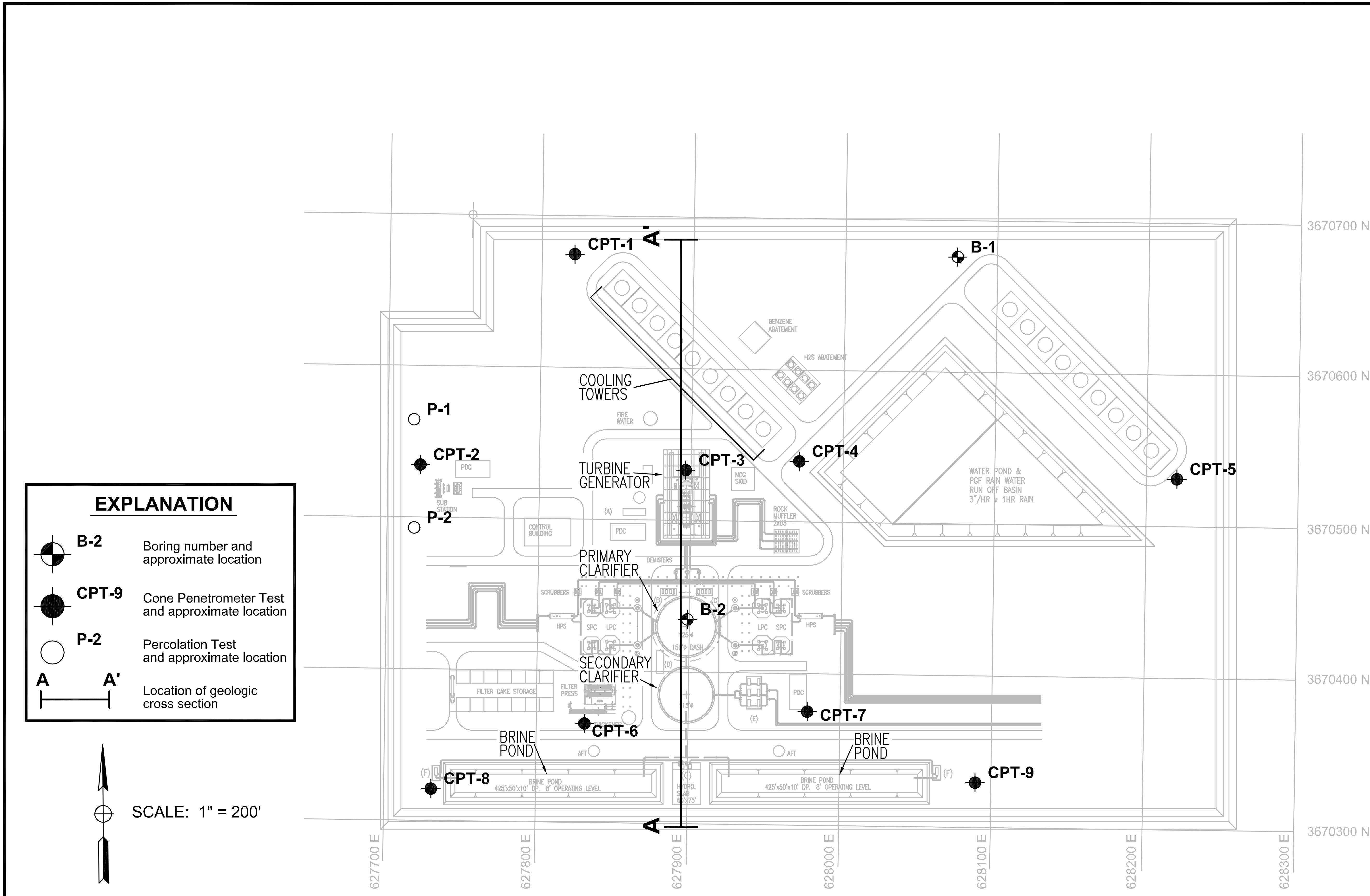
FIGURE 1b



Reference: Unit6 plotplan-2A.dwg, provided by Bibb and Associates, Inc.

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FIGURE 2

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SITE PLAN

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4.0 PROPOSED DEVELOPMENT

The proposed Salton Sea Unit 6 is a 180 megawatt, flash-steam geothermal power plant. The plant will use wells to extract the hot brine (about 300 to 330 ºF) that exists within the Salton Trough from depths of several thousand feet. Steam will be separated from the brine and used to drive a turbine generator. The used steam will then be condensed, cooled, and pumped back into the ground (along with the clarified brine) through the use of injection wells. The approximate locations and configurations of the proposed structures and improvements are shown on the Site Plan, Figure 2.

4.1 Facility

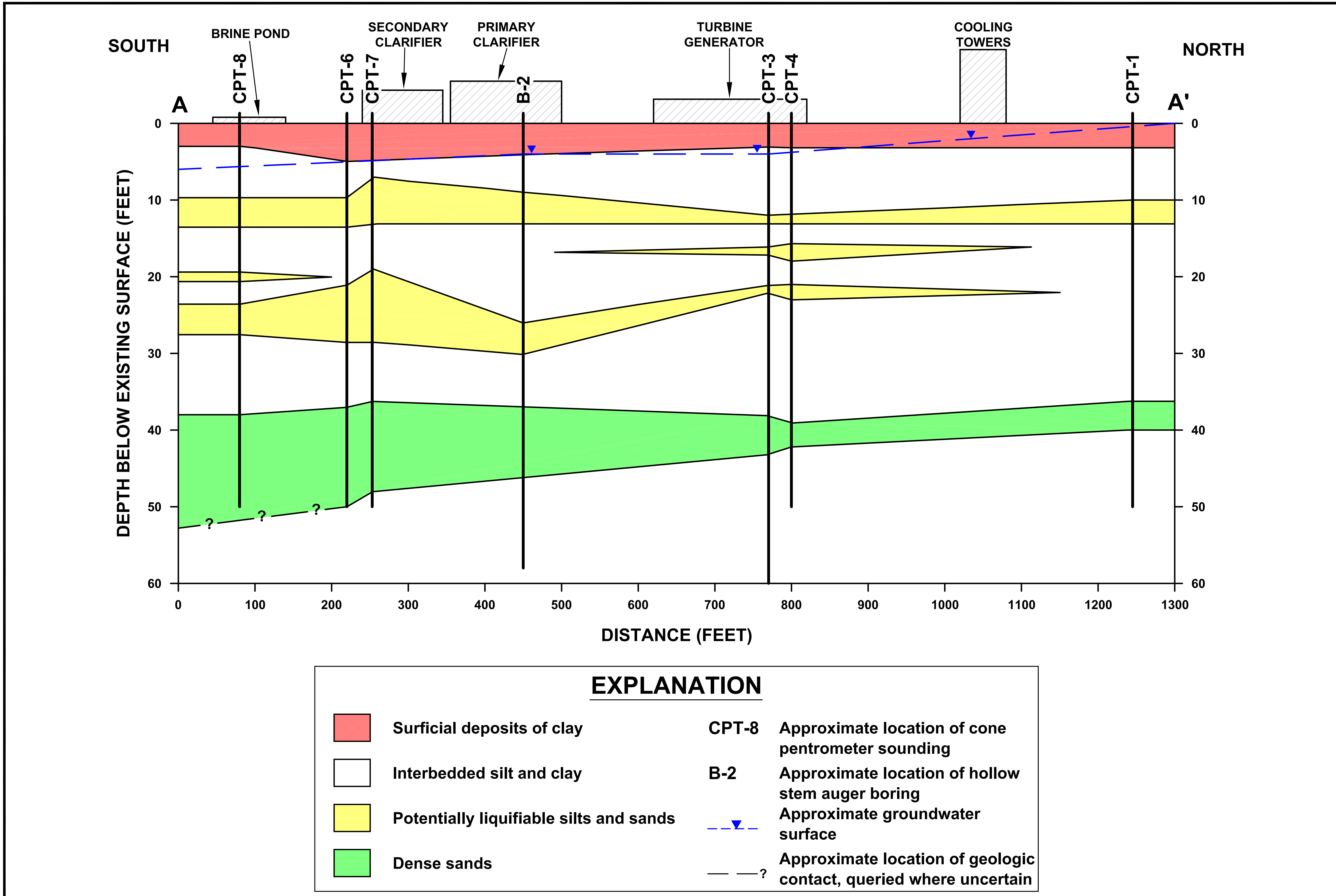
A variety of improvements will be associated with the proposed power plant. Above ground pipelines containing the hot brine will enter the site from off-site extraction wells. The high pressure steam (HPS) will be separated from the brine, passed through scrubbers and demisters, and sent into the turbine generator. Electricity from the generator will pass through high voltage overhead power lines through the grounding substation and out into the power network. The turbine will be overlain by a crane, underlain by steam condensers, and surrounded by a variety of associated improvements such as the lube oil and purge water pads. The condensed steam from the generator will be circulated through a series of cooling towers, and treated for benzene and hydrogen sulfide prior to being pumped back into the ground through off-site injection wells. The brine that does not produce steam will be passed through primary and secondary clarifiers, and pumped back into the ground through the off-site injection wells.

4.2 Brine Ponds

The brine generated from the extraction wells may occasionally need to be diverted to several brine ponds during servicing of the turbine. The brine ponds will be approximately 10 feet deep, and surrounded by 2:1 (horizontal:vertical) slopes. The brine ponds will be lined with high density polyethylene (HDPE) and covered with concrete. Evaporate from the ponds and/or sediment from the clarifiers will periodically be collected, passed through the filter press, and stored on site.

4.3 Dikes and Drainage

The site will be surrounded by an approximately 8 foot high earthen dike. The dike will be constructed using 2:1 side slopes. Note that the northern and western portions of the dike already exist, as described in Section 3.0. Minor grading will be necessary in order to provide level pads for the proposed improvements and drainage for surface water flow. Typical cut and fill depths of less than two to three feet are anticipated. The site will be



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GEOLOGIC CROSS SECTION

FIGURE 3

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graded to drain by sheet flow into an approximately five to six foot deep unlined detention basin. The detention basin will also be constructed using 2:1 slopes.

5.0 GEOLOGY AND SUBSURFACE CONDITIONS

The site is situated within the south-central portion of the Salton Trough, a topographic and structural depression bound to the north by the Coachella Valley and to the south by the Gulf of California. The Salton Trough is a region of transition from the extensional tectonics of the East Pacific Rise to the transform tectonic environment of the San Andreas system. Late Cenozoic extension associated with the opening of the Gulf of California formed this deep topographic and structural depression (Elders, 1972). The marine water of the gulf was cut off by growth of the Colorado River delta, resulting in the closed basin present today.

As the rift opened, the trough was filled with sediments derived from the adjacent mountains. The bottom of the trough is formed by Mesozoic and older metamorphic and crystalline rock. Tertiary marine and nonmarine deposits make up most of the estimated 15,000 feet of sediment which overlies the basement rock. The upper 3,000 feet of the Salton Trough is underlain by Pleistocene and Holocene alluvial and lacustrine (lake) deposits (Dibblee, 1954, Kovach, et al, 1962).

Volcanism and hydrothermal activity are the result of continuing extensional tectonics within the basin. There is a well defined, north-south trending zone of hydrothermal activity and volcanic rocks located along the axis of the trough, extending from the Salton Sea into Mexico. The hydrothermal activity is being used as a electrical power resource in the United States and Mexico.

5.1 Lacustrine Deposits

During the Late Pleistocene and Holocene, the basin was periodically inundated by floodwaters of the Colorado River to form lakes. Lake Cahuilla was formed during the last 1,000 years and evidence of its shoreline are still present around the Imperial Valley. The latest flooding, in 1905, created the present-day Salton Sea (Sharp, 1979).

The subject site is underlain by Holocene lacustrine deposits associated with ancient Lake Cahuillla. The lacustrine sediments are estimated to be roughly 100 to 300 feet thick (Kovach, et al). In general, the lacustrine deposits include sandy deltaic sediments, sandy beach deposits along ancient shore lines, and clay and silt in the middle of the ancient lake. The finer grained sediments contain lenses of sand toward the lake margins. The subsurface conditions in the upper 50 feet are illustrated schematically in the Geologic Cross Section, Figure 3. The approximate location of the cross section is shown on the Site Plan, Figure 2.

5.1.1 Lithology: The subsurface exploration indicates that the site is underlain by feet or more of lacustrine (lake) deposits. Based on the hollow stem borings and CPT interpretations, these deposits generally consist of relatively continuous thinly interbedded (1 to 4 feet thick) silty sand (Unified Soil Classification SM) and sandy silt (ML), with a few thin beds of clay (CL). All of the sediment was uniformly brown in color. Interpretation of the CPT logs indicates some channeling and cross-cut beds (Geologic Cross Section, Figure 3). There appears to be continuous, dense sand beds in the southern portion of the site. CPT soundings 5 through 9 suggest that dense sand beds are located between depths of approximately 20 and 28 feet below grade, and 36 and 48 feet below grade. The shallowest deposits generally consist of clays and include some vegetation from the farming operations.

The upper sandy deposits were typically loose to medium dense in consistency. The finer grained silts and clays were generally soft to firm in consistency, with low plasticity fines. Laboratory testing indicates that the surficial clays have a medium expansion potential (see Figure C-3 in Appendix C). The surficial soils are also relatively high in water soluble sulfates and chlorides (see Figure C-2).

5.1.2 Consolidation Characteristics: Laboratory consolidation tests were conducted on relatively undisturbed samples of the lacustrine deposits, as shown in Figures C-5.1 through C-5.4. These tests indicate that the clay beds are slightly overconsolidated, and moderately compressible (see Figure C-5.2). Because the area currently occupied by the Salton Sea was a dry lake bed for an extended period of time prior to the turn of century, it appears likely that the lacustrine sediments were overconsolidated due to lower groundwater levels in the past, and dessication of the surficial clays. The other consolidation samples were taken in silt and sand deposits, and appear to be disturbed as a result of the sampling procedure. However, these tests do suggest that the silts and sands are much less compressible than the clays. Time rate of consolidation measurements on the silts and sands agree with the findings of the CPT pore pressure dissipations, and indicate that the silts and sands will drain and settle rapidly after loading, and will serve to effectively drain the clay layers as well.

5.1.3 Percolation Characteristics: In order to aid in the design of an on-site septic disposal system, two percolation tests were conducted near CPT sounding 2. The percolation tests were conducted in general accordance with the requirements of the County of San Diego Department of Environmental Health. The approximate test locations are shown on the Site Plan, Figure 2. The test results are presented in Figure 4, and indicate that the soils in the proposed leach field location may percolate at a rate of between approximately 240 and 120 minutes per inch (or 1.3

PERCOLATION TEST RESULTS

TEST NO.	TIME INTERVAL [MIN]	WATER LEVEL DROP [IN]	PERCOLATION RATE [MIN/IN]	PERCOLATION RATE [GAL/DAY]
P-1	30 Minutes	1/4-Inch	120 Minutes/Inch	2.6 Gallons/Day
P-1	30 Minutes	1/8-Inch	240 Minutes/Inch	1.3 Gallons/Day
P-1	30 Minutes	1/4-Inch	120 Minutes/Inch	2.6 Gallons/Day
P-2	30 Minutes	1/4-Inch	120 Minutes/Inch	2.6 Gallons/Day
P-2	30 Minutes	1/4-Inch	120 Minutes/Inch	2.6 Gallons/Day
P-2	30 Minutes	1/8-Inch	240 Minutes/Inch	1.3 Gallons/Day

and 2.6 gallons per day). Note that pore pressure dissipations and field observations indicate that groundwater exists near the surface elevations in this area of the site.

5.1.4 Resistivity Characteristics: In order to aid in the design of the grounding system for the power plant, a resistivity survey was conducted at the location of the proposed substation. The resistivity survey was conducted for variable depths up to 50 feet, in a line that was oriented approximately north to south. The Wenner four pin survey was conducted in general accordance with ASTM G-57 and IEEE 81 by M. J. Schiff and Associates on December 4, 2001. The test results and interpreted variations in resistivity with depth are presented in Appendix F.

5.2 Groundwater

Groundwater was encountered at relatively shallow depths across the site. Groundwater was observed at a depth of 4 feet in the hollow stem borings. The pore pressure dissipations taken on the CPT soundings indicate that groundwater levels generally range from zero to 6 feet below existing grade throughout the site. Note that artesian flow was observed in CPT-2 near the Vail Lateral 5 Drain along the western edge of the site. The artesian condition appears to be the result of the higher elevation of water in the Vail Lateral Drain. In general, the groundwater elevations are believed to be associated with both irrigation water and the Salton Sea. A recent survey of the site indicates that the water in the Vail Lateral Drain is about 8 feet above the lowest portions of the site (which are at minus 232 feet), and that the Salton Sea level is about 5 feet higher than the lowest portions of the site. We anticipate that groundwater depths deepen to the south and east. The approximate average groundwater depths are depicted on the Geologic Cross Section, Figure 3.

6.0 GEOLOGIC HAZARDS AND SEISMICITY

The primary geologic hazards at the site include strong ground motion from a seismic event centered on one of several nearby active faults, and liquefaction of the sandy soils which underlie the site given strong ground shaking. In addition, there is the potential for earthquake induced flooding of the site if the Vail Lateral 5 Drain along the western property line were to fail. However, evaluation of the stability of the dike was outside of the scope of services provided for this investigation. Additional subsurface investigation, soil sampling, and slope stability analysis will be necessary to provide an opinion regarding the stability of the existing dike. The general geologic hazards and seismicity are described in greater detail below.

6.1 Regional Seismicity

The subject site is located within one of the most seismically active areas in California. The regional seismicity is depicted on the Fault Location Map, Figure 5. In general, the Salton Trough is the zone of transition between the ocean floor spreading regime in the Gulf of California, and the right-lateral, strike-slip regime of the San Andreas Fault system. A list of active faults within 100 km of the site is presented in Table 1.

6.2 Ground Rupture

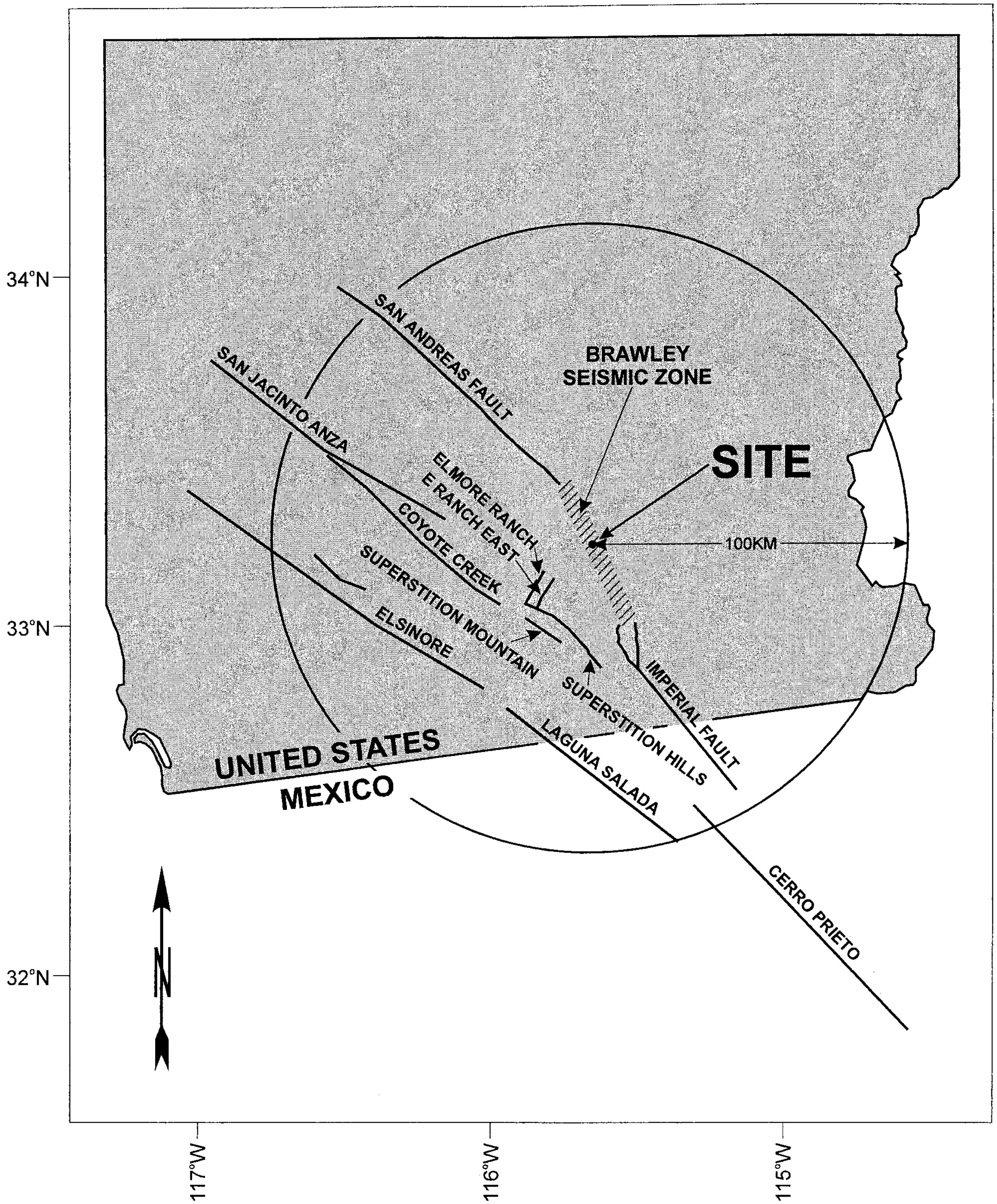
Ground rupture occurs when motion on an active fault reaches the ground surface. The site is situated within the Brawley Seismic Zone, which is a zone of transition between the northwest end of the Imperial fault and the southwest end of the San Andreas fault. The zone is defined by epicenters of microseismic events or aftershocks following earthquakes on the Imperial fault, rather than geologic mapping of surface ruptures. There are no known faults within the zone that reach the ground surface, and the site is not within a Alquist-Priolo Special Studies Zone (Hart, 1990). It is likely that fault slip within this zone is being translated into regional subsidence and geothermal activity (see Section 6.7). The potential for ground rupture due to faulting is believed to be low. Ground rupture due to severe shaking is possible. However, the recommended improvements should reduce the impacts.

6.3 Ground Motion

Several commercially available computer programs were used to evaluate potential ground motions at the site, including EQSEARCH, EQFAULT and FRISKSP (Blake, 1998). The results of our analyses are presented in Appendix D. According to the program 3-D TopoQuads, the subject site is located approximately at a latitude of 33.168º north, and a longitude of 115.619º west. The Fault Location Map (Figure 5) shows the known active faults within a 100 km radius of the site. Table 1 summarizes the properties of these faults.

6.3.1 Historical Seismicity: The historical seismicity at the site was evaluated using the program EQSEARCH (Blake, 1998). The EQSEARCH database contains the locations, dates, and magnitudes of recorded earthquakes within 100 km of the site through the year 1998. The seismicity data is presented in Appendix D, along with the Seismic Recurrence Curve, and a map showing epicenters and magnitudes.

6.3.2 Deterministic Analysis: A deterministic analysis was conducted using the program EQFAULT. The estimated "maximum credible" peak ground accelerations from faults within 100 km of the site are shown in Table 1. The attenuation relationship used for the analysis was based on seismic data taken from deep soil



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FAULT LOCATION MAP

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FIGURE 5

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FAULT[1]	DISTANCE TO SITE [KM]	MAX. CRED.[5] DETERMINISTIC PGA[2]	MAX. CRED.[5] MOMENT MAGNITUDE[3]	ESTIMATED FAULT AREA[4] [CM^2]	ESIMATED SHEAR MOD.[4] [DYNE/CM^2]	ESIMATED SLIP RATE[4] [MM/YEAR]
Brawley Seismic Zone[6]	<2	0.46	6.4	1.20E+13	3.30E+11	1.80
Elmore Ranch (East)	16	0.14	6.1	1.11E+12	3.30E+11	1.50
Elmore Ranch (West)	17	0.15	6.2	1.32E+12	3.30E+11	1.50
San Andreas (Coachella Valley)	22½	0.20	7.1	1.14E+13	3.30E+11	25.00
San Jacinto (Superstition Hills)	24½	0.14	6.6	3.89E+12	3.30E+11	4.00
Imperial-Brawley	26½	0.16	6.9	7.92E+12	3.30E+11	20.00
San Jacinto (Superstition Mountain)	30½	0.09	6.3	1.68E+12	3.30E+11	5.00
San Jacinto (Coyote Creek)	35½	0.13	7.1	1.14E+12	3.30E+11	4.00
Elsinore	56½	0.06	6.8	3.00E+13	3.30E+11	5.00
Laguna Salada (Mexico)	57½	0.08	7.1	1.12E+13	3.30E+11	3.50
Cerro Prieto (Mexico)	88	0.05	7.1	1.16E+13	3.30E+11	20.00

1. Fault activity determined by Blake (1998), CDMG (1992), Wesnousky (1986), and Jennings (1975).
2. Peak horizontal ground accelerations from Sadigh (1997) for Deep Soil Sites for the Maximum Credible Earthquake.
3. Magnitudes determined from Blake (1998), OSHPD (1995), Mualchin and Jones (1992), Wesnousky (1986) and Anderson (1984).
4. Estimated fault area, shear modulus, and slip rate after fault data for EQFAULT and FRISKSP, Blake (1998).
5. The Maximum Credible Earthquake is defined as the *median* maximum earthquake that appears capable of occuring under the known tectonic framework, according to the relationship ($Mw = 4.09 + 0.98*LOG A$).
6. The estimated Maximum Credible Magnitude for the Brawley Seismic Zone is from EQFAULT, Blake (1998).

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REGIONAL SEISMICITY

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TABLE 1

sites (Sadigh, 1997). According to the deterministic analysis, a peak ground acceleration (PGA) of 0.46g would occur at the site if a moment magnitude 6.4 earthquake occurred on the Brawley Seismic Zone.

It should be noted that the estimated "maximum credible" peak ground accelerations from a deterministic analysis are often mistakenly believed to represent the "worse-case" ground motions. However, deterministic analyses generate ground motions that would occur if the entire fault area in question were to rupture with the associated *median* magnitude (this is commonly termed the "maximum credible" magnitude, even though seismological data indicates that larger magnitude events would occur about one half of the time given the same rupture area). Deterministic analyses also typically use the *median* values from the selected attenuation relationship in order to estimate accelerations from associated magnitudes and distances. At the subject site, the deterministic ground motions are lower than the probabilistic results described below because the statistical variations in magnitude and attenuation are not included in the deterministic analyses.

6.3.3 Probabilistic Analysis: In order to provide an estimate of the potential peak ground acceleration that structures founded at the site may experience in time, the program FRISKSP was used perform a probabilistic analysis of seismicity. The probabilistic analysis incorporates the contribution of all known active faults within a 100 km radius of the site for which published fault data is available. The analysis attempts to account for uncertainty in rupture size, rupture location, magnitude and frequency, as well as uncertainty in the attenuation relationship. The probabilistic analysis was conducted using the characteristic earthquake distribution of Youngs and Coppersmith (1985), with the same attenuation relationship for deep soil sites used for the deterministic analysis (Sadigh, 1997).

Based on the results of the probabilistic analysis, the Upper Bound Earthquake for the site, defined as the motion having a 10 percent probability of being exceeded in a 100 year period, results in a peak ground acceleration (PGA) of 1.35g. The Upper Bound Earthquake has an associated return period of roughly 1,000 years. The Design Basis Earthquake results in a PGA of 1.16g. The Design Basis Earthquake is estimated to have a 10 percent probability of being exceeded in 50 years (or a 475 year return period). For liquefaction analysis, the PGA value should incorporate magnitude weighting in order to normalize all of the seismic events with respect to the magnitude 7.5 event used to develop the published liquefaction relationships (SCEC, 1999). The magnitude weighted Design Basis PGA for use in liquefaction analysis is 0.92g.

6.4 Liquefaction

Liquefaction is a process in which soil grains in a saturated sandy deposit lose contact due to earthquakes or other sources of ground shaking. The soil deposit temporarily behaves as a viscous fluid; pore pressures rise, and the strength of the deposit is greatly diminished. Liquefaction is often accompanied by sand boils, lateral spread, and post-liquefaction settlement as the pore pressures dissipate. Liquefiable soils typically consists of cohesionless sands and silts that are loose to medium dense, and saturated. Clayey soils do not liquefy because the soil skeleton is not supported by grain to grain contact, and is therefore not subject to densification by shaking.

6.4.1 Historical Liquefaction: The site is located within an area which has previously been shown as potentially susceptible to liquefaction. Liquefaction during earthquakes on the Imperial fault was widespread in Imperial County. The occurrences were typically located in river drainages or adjacent to canals, and in delta or shoreline facies of Lake Cahuilla deposits. The liquefiable sites contained predominately loose sandy soils, or sequences of thick sandy layers within finer grained soils (Youd and Wieczorck, 1982, Holtzer et al., 1989). Although no indications of previous liquefaction such as sand boils were observed at the subject site, such features would likely have been obscured by farming operations.

6.4.2 Liquefaction Analysis: Liquefaction analysis was performed on data from the CPT soundings using the most recent advances to the simplified method of analysis (Youd et al, 2001). The results are presented along with a brief overview of the analysis in Appendix E. Our analysis indicates that liquefaction of some of the sandy deposits up to about 40 feet deep is likely given relatively low levels of ground shaking such as 0.2 to 0.3g. Note that the magnitude weighted Design Basis peak ground acceleration from the probabilistic analysis is 0.92g. At this level of ground shaking, much of the sandy deposits at the site will probably liquefy. However, the dense sand beds that are situated between depths of 36 and 48 feet (see Section 5.1) do not appear to be liquefiable given the Design Basis ground shaking (see Figures E-5.1 through E-9.2). These deposits generally have an average normalized tip resistance $(q_{c1N})_{cs}$ in excess of 160, and may for practical purposes be considered nonliquefiable (SCEC, 1999). These dense sands would therefore be a suitable bearing layer for settlement sensitive structures founded on driven piles.

6.4.3 Post-Liquefaction Settlement: Liquefaction is commonly followed by settlement as the excess pore pressures dissipate and the sand grains redistribute stresses. Post liquefaction settlement at the site was estimated using procedures that may be applied to CPT soundings (Ishihara, 1996). The settlement analysis was conducted for each CPT sounding, with all of the loose sands completely liquefied from the Design Basis Earthquake. Our analysis indicates that post-liquefaction settlement at the site may vary from 6 to 9 inches. According to state guidelines, a differential settlement equal to one-half (or less) of the anticipated total liquefaction settlement may be conservatively assumed for relatively flat sites (SCEC, 1999). Based on these guidelines, and the observed variations in total settlement from the different CPT soundings, we anticipate that differential settlement across the site from complete liquefaction may typically be on the order of 3 to 4 inches. Differential settlements across smaller structures may be less than this amount.

6.5 Landslides and Lateral Spreads

Evaluation of the stability of the earthen dikes along the northern and western edges of the site was outside of the scope of services provided for this investigation. Other than these dikes, the subject site is nearly flat-lying, and there are no areas of sufficient relief to cause landsliding or lateral spreading. Our analysis indicates that the proposed permanent 2:1 slopes for the brine ponds and detention basin should be stable. During construction, temporary slopes should be stable provided that the excavations are dewatered and/or do not extend to depths where heavy seepage is encountered. Shallow failures on the side walls of the detention basin may be possible given sufficient long term seepage. In general, within the areas of our investigation, the potential for landsliding or lateral spreading to adversely impact the proposed improvements is considered to be low. However, we should reiterate that the stability of the exiting dikes is not known at this time.

6.6 Tsunamis, Seiches, Earthquake Induced Flooding

There are a variety of large bodies of water which could flood the site as the result of earthquake induced ground motion. These include tsunamis within the Gulf of California, seiches within the Salton Sea, and flooding from a failure on the Vail Lateral 5 Drain. However, the proposed earthen dikes which will surround the site may help to reduce the potential for flood damage. Each hazard is discussed in greater detail below.

6.6.1 Tsunamis: The site is situated several hundred feet below sea level. This suggests that the potential may exist for inundation in the event of a tsunami within the Gulf of California. However, the configuration of the Gulf of California, and the higher ground surface elevations near Calexico, have historically provided relief from such events. There are no records which indicate that tsunamis have impacted the Imperial Valley in the last several hundred years. Therefore, the potential for a tsunami to impact the site is considered to be low.

6.6.2 Seiche: A seiche is a wave in an enclosed body of water created by earthquake shaking. The potential for a seiche to occur is related to the natural frequency of vibration of the body of water, as well as the predominate frequencies of vibration in the seismic event. The possibility may exist for a seiche to occur in the Salton Sea. It is our understanding that seiching has not been observed in recent seismic events in the Imperial Valley. However, because the site is situated below the level of the Salton Sea, and because the western dike has only a few feet of freeboard, it is our opinion that seiche induced flooding of the site has a moderate potential.

6.6.3 Earthquake Induced Flooding: The most likely cause of earthquake induced flooding at the site would be associated with failure of the existing Vail Lateral 5 Drain embankment which borders the western edge of the site. Evaluation of the stability of the dike was outside of the scope of services provided for this investigation. However, it should be noted that seismic slope failure or liquefaction settlement could result in flooding of the proposed improvements.

6.7 Subsidence

The regional faulting regime in the Salton Trough may be described as a graben, or a fault bounded basin. The area is subsiding due to regional faulting faster than the sediments are accumulating. Extraction of groundwater may also be contributing to regional subsidence. CalEnergy has been surveying the site vicinity for many years. It is estimated that tectonic subsidence and fluid withdrawal combine for roughly 4 centimeters of subsidence each year (Lofgren, 1978). Subsidence due to tectonics and fluid withdrawal is believed to be occurring over a very large area. Consequently, the potential for damaging differential settlement resulting from tectonic and fluid withdrawal subsidence is probably low. The largest hazard to the site from subsidence is the rising level of the Salton Sea.

7.0 CONCLUSIONS

There were no geotechnical conditions apparent during the investigation which would preclude the proposed development. However, several geotechnical risks exist which require special consideration in order to reduce the potential for distress to the proposed improvements.

- The site is situated in a zone of high seismic activity. The Brawley Seismic Zone directly underlies the proposed improvements. The potential exists for strong ground shaking from a variety of nearby seismic sources, including the Brawley Seismic Zone, the San Andreas Fault and the Imperial Fault. This hazard is generally mitigated through seismic design in accordance with the applicable building codes. It is our opinion that the potential for ground rupture from faulting in the Brawley Seismic Zone is low. Tectonic subsidence of 4 cm per year should be allowed for in design of dikes around the development.

- The site is underlain by deep lacustrine deposits, which include loose silty sands, and soft silts and clays. Our analysis indicates that liquefaction of the loose sand and silt deposits is likely in the event of strong ground shaking from one of the many nearby seismic sources. Post liquefaction settlement of between 6 and 9 inches is estimated. In order to reduce the liquefaction potential, ground improvement is recommended beneath all settlement sensitive improvements to a depth of 40 feet. Ground improvement alternatives include the use of earthquake drains, terra-probing, vibro-flotation, or vibro-replacement (stone columns). Additional subsurface investigation will be necessary after ground improvement operations are completed in order to confirm that the liquefaction hazard has been mitigated.

- The lacustrine deposits are susceptible to settlement from foundation or fill loads. For heavily loaded settlement sensitive structures where less than ½ inch of total settlement is required, deep foundations (driven piles) will be necessary in addition to ground improvement. In order to reduce differential settlements to within tolerable limits for the minor structures, surcharge loads, and/or the removal and compaction of the soil underlying the foundations will be necessary. Estimated settlements for various foundation loading conditions and remedial grading options are presented in the foundation section of this report. These estimates are based on unimproved soil conditions. Surcharge loading or ground improvement will reduce the estimated settlements. Additional subsurface investigation after ground improvement and surcharging could be used to confirm that the ultimate settlement potential has been reduced to within tolerable limits.

- We recommend that the deep foundations for settlement sensitive improvements be driven into the dense sands that exist between 36 and 48 feet below grade. In order to reduce the potential for damaging the piles during driving through the upper sand beds, the piles should be pre-drilled to a depth of 35 feet, using hollow stem augers that will create holes of no more than 75 percent of the area of the precast, square concrete piles that will be used. The holes should be kept full of water or drilling mud in order to reduce the potential for caving.

- The dense sands that should support the pile tips exist primarily in the southern and eastern portions of the site. In the area of the proposed turbine generator, these sand beds are thinner, and may not be penetrated during driving. Indicator piles and pile load tests should therefore be conducted prior to construction in order to verify that the sands exist at the proposed pile tip elevations in the northern and western portions of the site. Deeper piles may be necessary in these areas if the sands are found to be discontinuous. Alternatively, the layout of the plant may be modified so that the settlement sensitive improvements are situated in the southern and eastern portions of the site.

- Groundwater was encountered at depths of between zero and six feet below existing grades. The site is generally situated below the Salton Sea, and heavy groundwater seepage may be encountered in excavations for the brine ponds and the retention basin, as well as during any remedial earthwork conducted beneath the proposed improvements. Dewatering of these excavations may be accomplished through the use of sumps or well points. Permanent dewatering of the brine ponds and retention basin may also be necessary. Alternatively, the concrete lined brine ponds may be designed to resist the uplift pressures exerted by the groundwater seepage. This may include the use of anchor piles.

- The subsurface investigation indicates that the surficial soils at the site typically include between 2 and 6 feet of saturated lean clay. Percolation tests in the clay suggest that seepage rates in the area of the proposed septic disposal system may vary between approximately 1.3 and 2.6 gallons per day. This may not provide suitable drainage for the proposed septic system. Alternative locations or configurations should be considered.

- The potential may exist for earthquake induced flooding given a seismic slope failure on the Vail Lateral 5 Drain. Evaluation of the stability of this earthen structure was outside of the scope of services provided for this investigation. Additional subsurface investigation and slope stability analysis should be conducted in order to properly characterize this hazard.

8.0 RECOMMENDATIONS

The remainder of this report presents recommendations regarding earthwork construction and foundation design. These recommendations are based on empirical and analytical methods typical of the standard of practice in southern California. If these recommendations appear not to cover any specific feature of the project, please contact our office for additions or revisions.

8.1 Plan Review

We recommend that foundation and grading plans be reviewed by Geotechnics Incorporated prior to finalization in order to evaluate conformance with the intent of the recommendations contained within this report. Any changes to the locations of the improvements shown on the Site Plan may require additional subsurface investigation.

8.2 Excavation and Grading Observation

Site grading, ground improvement, pile driving, and foundation excavations should be observed by Geotechnics Incorporated. Geotechnics Incorporated should provide observation and testing services continuously during grading, ground improvement, and pile driving operations. Such observations are considered essential to identify field conditions that differ from those anticipated by the preliminary investigation, to adjust designs to actual field conditions, and to determine that these operations are accomplished in general accordance with the recommendations of this report. Recommendations presented in this report are contingent upon Geotechnics Incorporated performing such services. Our personnel should perform sufficient testing of fill during grading to support our professional opinion as to compliance with compaction recommendations.

8.3 Earthwork

Grading and earthwork should be conducted in accordance with the recommendations in this report, and the requirements of the California Building Code, the County of Imperial and other regulatory agencies. The following recommendations are provided regarding specific aspects of the proposed earthwork construction. These recommendations should be considered subject to revision based on field conditions observed by the geotechnical consultant.

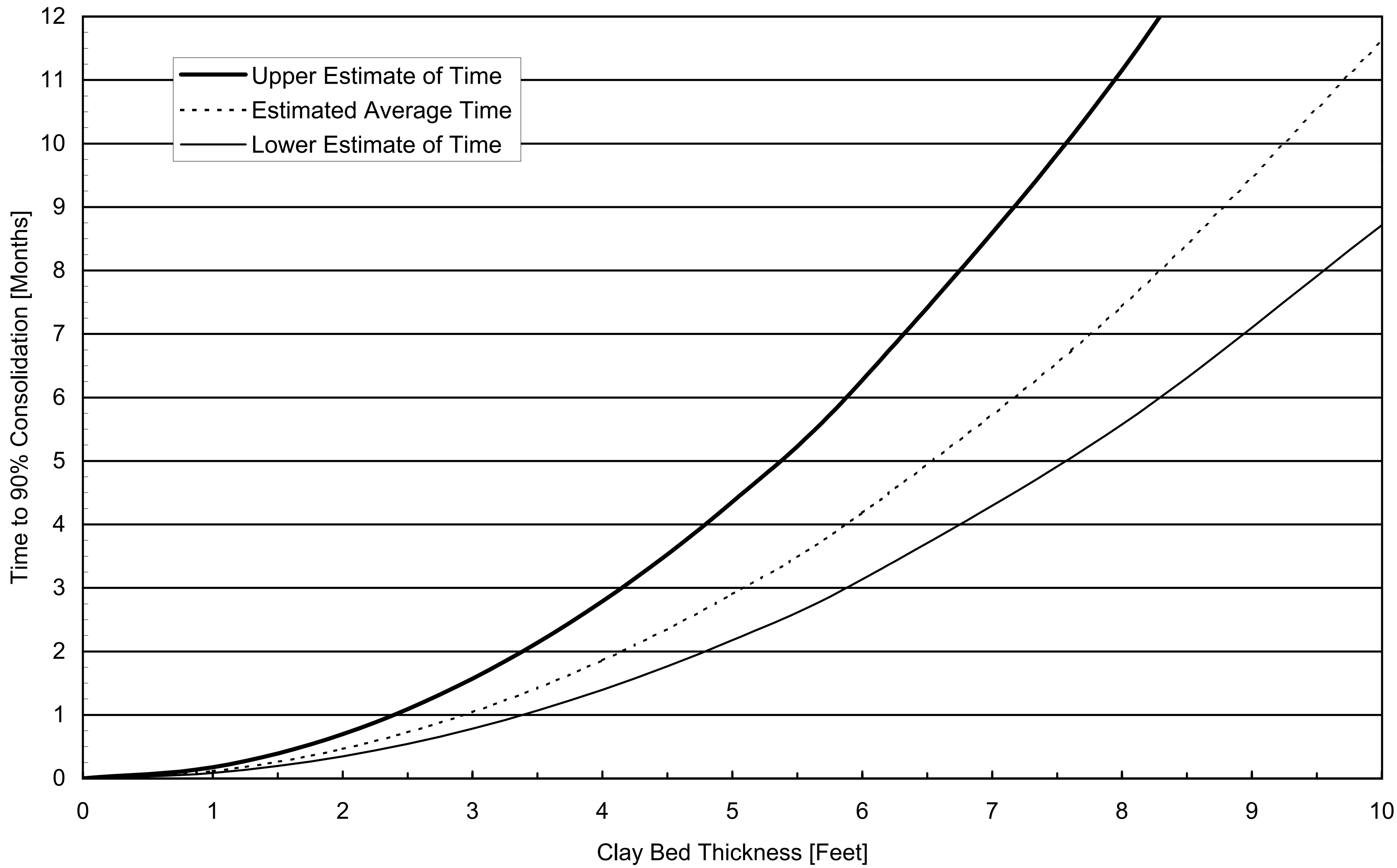
8.3.1 General: Site preparation should include the removal of deleterious materials, existing structures, or other improvements from areas to be subjected to fill or structural loads. Deleterious materials include vegetation, trash, debris, and rock fragments with greatest dimensions in excess of 6 inches. These materials may be stockpiled in non-structural portions of the site, or exported to a legal landfill.

8.3.2 Improvement Areas : The surficial soils throughout the site have been used for agricultural operations. These materials are not suitable for the direct support of fill or foundation loads. Consequently, in all areas of planned improvements, including fills, pavements, exterior flatwork, and structures, the surficial 12 inches of soil should be excavated and stockpiled. The exposed materials should be scarified to a depth of 12 inches, brought to slightly above optimum moisture content, and compacted to at least 90 percent relative compaction as described in Section 8.3.8. The stockpiled soils that are freed of deleterious materials may then be replaced as a uniformly compacted fill to the proposed finish grades. Additional remedial grading may be necessary for structures, as discussed in Sections 8.3.3 and 8.3.4.

8.3.3 Surcharge Loads : The lacustrine soils underlying the site are compressible, and will settle due to the proposed foundation loads. The amount of settlement will depend upon the magnitude of the foundation loads, as discussed in Section 8.4. However, the ultimate foundation settlement may be reduced by surcharging the foundation area prior to construction. For example, we estimate that a 2 foot thick fill load over a large area (a radius of 100 feet or more) will result in 1 to 2 inches of total settlement. Larger fill loads will induce more settlement. The fill load could therefore be designed to reduce the foundation settlement to within tolerable limits.

Laboratory testing indicates that the sands and silts will settle almost immediately upon loading. The time for settlement to be completed will therefore be controlled by the thickest clay beds underlying the surcharge area. The estimated range of times for settlement of clay beds with various thicknesses are shown in Figure 6. Because the clay beds are generally less than 5 feet thick, we estimate that settlement from surcharge fill or foundation loads should generally be completed at the site in under 2 to 4 months. If surcharge loading is used, the surcharged area should be instrumented with survey monuments to confirm that settlement is completed, as shown in Figure 7.

8.3.4 Building Areas (Minor Structures) : There are two geotechnical conditions within all of the building areas which should be addressed: soil settlement from static foundation loads, and post-liquefaction settlement. The static settlement potential for lightly loaded minor structures (such as the Power Distribution Center or the Control Building) may be reduced by surcharging as described in Section 8.3.3, and/or by over-excavating and compacting the soils directly beneath the foundations, as described in Section 8.4. We recommend that a surcharge and remedial grading program be designed for each minor structure (based on the actual dead and live foundation loads) in order to reduce static soil settlement to within tolerable limits.



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TIME TO 90% CONSOLIDATION

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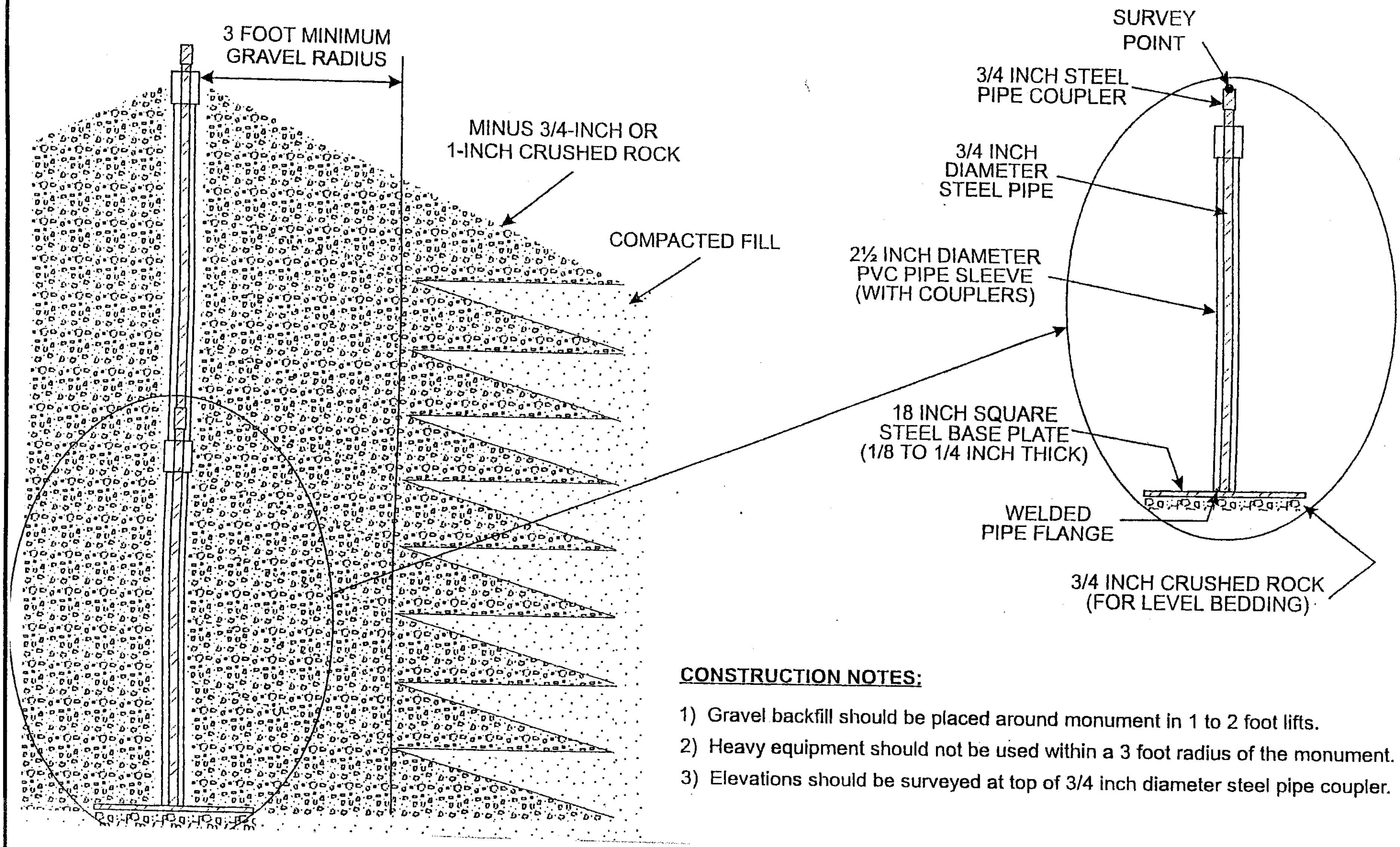
FIGURE 6



CONSTRUCTION NOTES:

1) Gravel backfill should be placed around monument in 1 to 2 foot lifts.
2) Heavy equipment should not be used within a 3 foot radius of the monument.
3) Elevations should be surveyed at top of 3/4 inch diameter steel pipe coupler.

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SETTLEMENT MONUMENTS

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FIGURE 7

However, surcharging and remedial grading will not significantly reduce the estimated post-liquefaction differential settlement.

If the potential for post-liquefaction settlement of 3 to 4 inches is deemed intolerable to the minor structures, then ground improvement should be conducted. Ground improvement should extend down to below the depth of the potentially liquefiable sand and silt deposits (which is generally about 40 feet below existing grades). Ground improvement alternatives include the use of earthquake drains, terra-probing, vibro-flotation, or vibro-replacement (stone columns). Earthquake drains provide a relief for excess pore pressures, and provide some densification from vibrations during installation. Terra-probing consists of advancing a large diameter steel casing to the required depth by vibrating. Vibro-flotation (also called vibro-compaction) uses vibrating probes as well. In both cases, the vibrations densify only the sandy and silty soils (which are the soils that are susceptible to liquefaction).

Vibro-replacement (stone columns) is generally more effective at improving fine grained or interbedded soil deposits, such as those encountered at the subject site. In the vibro-replacement process, a crane mounted vibrator would penetrate the soil to approximately 40 feet below the existing ground surface. As the vibrator is withdrawn, crushed rock backfill would be added in compacted lifts to displace the surrounding soil and form the stone column. We recommend that 2 to 3 foot diameter columns be constructed on a maximum spacing of 6 to 8 feet, center to center. The stone backfill should have a minimum friction angle of 40 degrees. The stone columns should extend out beyond the edge of the structures for at least two additional rows. A specialty contractor should be contacted to provide more specific construction recommendations and details.

The construction of stone columns increases soil density through displacement of the existing materials as well as the effects of vibration. Stone columns will effectively reduce the potential for settlement, increase the bearing capacity of the saturated lacustrine deposits, and reduce the potential for liquefaction by both increasing the density of the underlying sand beds and providing a relief for excess pore pressure.

8.3.5 Building Areas (Major Structures): It is our understanding that the major structural improvements (such as the turbine generator and clarifier complex) will need to be designed to limit total settlement to less than ½ inch. Consequently, we recommend that these improvements be founded on driven piles. The piles should be extended below the liquefiable sand and silt deposits, and into the dense sands. Based on the existing subsurface data, we anticipate that the piles would need to be driven approximately 35 to 45 feet below existing soil grade in order to encounter the dense sands. To reduce the potential for damaging the piles during driving through the interbedded clays and sands, the piles should be pre-drilled in the upper 35 feet with water or mud filled hollow stem augers. The diameter of the auger should be the same as the side length of the square concrete piles that will be used. Additional recommendations for the design of the pile foundations are provided in Section 8.4.

Pile foundations will effectively reduce the potential for settlement due to large static loads. However, piles will not eliminate the potential for liquefaction. Post-liquefaction settlement would result in down-drag on the piles. The resulting increase in vertical load could result in punching of the pile tips through the bearing sands, and excessive settlement. Furthermore, the lateral load carrying capacity of the piles would be greatly reduced during liquefaction, and large lateral displacements could occur. Finally, the post-liquefaction soil settlement under the structure could still cause substantial damage to the slabs-on-grade between the piles or to associated utilities. In order to reduce the potential for these problems, ground improvement should be conducted in addition to the use of deep foundations. The ground improvement alternatives described in Section 8.3.4 for minor structures are also appropriate for the major improvements. However, it should be noted that if stone columns are used, the column locations will need to be carefully coordinated with the pile foundation layout, in order to avoid driving piles through the columns. Once again, we recommend that a specialty contractor be contacted in order to provide more specific construction recommendations and details.

8.3.6 Temporary Excavations: Temporary excavations may be necessary to complete remedial earthwork conducted for the minor structures (see Section 8.3.4). Temporary excavations deeper than a few feet will likely encounter groundwater seepage, and will need to be dewatered as discussed in Section 8.3.7. Temporary excavations in dewatered areas should be inclined no steeper than 1:1 for heights up to 10 feet. Deeper excavations should be evaluated on a case-by-case basis.

8.3.7 Dewatering: Any excavations on site of more than a few will likely encounter groundwater seepage, and will need to be dewatered. Dewatering wells should be situated at regular intervals outside of the excavation areas, and should be extended to a depth below the excavation bottoms equal to at least 80 percent of the height of water above the excavation bottoms. For example, an excavation that extends 10 feet below the ground water level should be dewatered to 8 feet below the excavation bottom, or 18 feet below the groundwater level. If water accumulates in the excavations, dewatering may also include the use of sumps. A specialty contractor should provide specific dewatering recommendations.

It should be noted that the use of heavy equipment in close proximity to groundwater or saturated soils may create unstable subgrade conditions. In this event, light equipment should be used to smooth the excavation bottom, and a stress absorbing geogrid such as Tensar BX1100 should be placed on the excavation bottom. One to two feet of minus ¾-inch crushed rock should then be placed over the geogrid until the surface is stabilized (the necessary thickness of rock for stabilization may be determined by observation). Note that various geotextiles can also be used to stabilize excavations, but more crushed rock or base will typically be required.

8.3.8 Fill Compaction: All fill and backfill to be placed in association with site development should be accomplished at slightly over optimum moisture conditions and using equipment that is capable of producing a uniformly compacted product. The minimum relative compaction recommended for fill is 90 percent of maximum density based on ASTM D1557-91. Sufficient observation and testing should be performed by Geotechnics Incorporated so that an opinion can be rendered as to the compaction achieved.

Imported fill sources, if needed, should be observed prior to hauling onto the site to determine the suitability for use. Representative samples of imported materials and on site soils should be tested by Geotechnics in order to evaluate their appropriate engineering properties for the planned use. Imported fill soils should have an expansion index of no more than 50 based on ASTM D4829.

During grading operations, soil types other than those analyzed in the geotechnical report may be encountered by the contractor. Geotechnics should be notified to evaluate the suitability of these soils for use as fill and at finish grade.

8.4 Foundation Recommendations

The following recommendations are considered generally consistent with methods typically used in southern California. Other alternatives may be available. The foundation recommendations herein should not be considered to preclude more restrictive criteria of governing agencies or the structural engineer. The design of the foundation system should be performed by the project structural engineer. The shallow foundation alternative is considered to be appropriate for minor structures with or without ground improvement. The deep foundation recommendations are considered to be appropriate for major structures with ground improvement.

8.4.1 Minor Structures: Shallow foundations may be appropriate for relatively lightly loaded minor structures at the site, such as the Power Distribution Center and Control Building. In order to reduce settlement of these structures to within tolerable limits, surcharge loading and/or remedial excavations may be conducted, as discussed in Sections 8.3.3 and 8.3.4. Surcharge loads and remedial excavations should generally extend at least 5 feet beyond the structural perimeter. A range of estimated settlement for various foundation configurations and loads is presented in Section 8.4.1.3. An appropriate surcharging and/or remedial grading program may be developed for each minor structure based on the tolerable settlements, and the anticipated maximum dead and live loads. Note that if ground improvement is conducted for the minor structures, then the ultimate settlements estimated in Section 8.4.1.3 will be further reduced.

8.4.1.1 Foundations : The following criteria are provided for light structures founded on firm and unyielding soils. Note that the allowable soil bearing has been lowered to reduce the estimated settlements.

Allowable Bearing Capacity:	1,500 lbs/ft^2 gross (allow a one-third increase for short-term wind or seismic loads)
Minimum Footing Width:	18 inches
Minimum Footing Depth:	24 inches below lowest adjacent soil grade.
Minimum Reinforcement:	Two No. 5 bars at both top and bottom, or as recommended by the structural engineer.
Slabs-on-Grade:	Slabs should be at least 6 inches thick, and reinforced with at least No. 3 bars on 18-inch centers, each way.

8.4.1.2 Lateral Loads : Lateral loads for minor structures may be resisted by friction between the bottoms of the shallow foundations and slabs and the supporting soil, as well as passive pressure from the portion of vertical foundation members embedded into the soil. A coefficient of friction of 0.25 and a passive pressure of 300 lb/ft^3 is recommended.

8.4.1.3 Settlement : Settlement may result from the application of static fill or foundation loads, as well as from liquefaction of the sandy lacustrine soils. We estimated that post-liquefaction differential settlement will be on the order of 3 to 4 inches, as discussed in Section 6.3. If the potential for this settlement is intolerable, then ground improvement should be conducted.

Settlement analysis was conducted for various foundation and fill loading conditions in order to provide general guidelines for remedial grading and surcharging. The results of the analyses are summarized in Figure 8. The analyses were based on the results of consolidation testing of undisturbed samples of the lacustrine deposits from Boring 2 (see Figures C-5.1 through

SURCHARGE FILL LOADS[1]

LOAD CONDITION	MINIMUM SETTLEMENT [IN]	MAXIMUM SETTLEMENT [IN]	MAX. SETTLEMENT WITH 2 FOOT O.X. [IN]	MAX. SETTLEMENT WITH 4 FOOT O.X. [IN]	MAX. SETTLEMENT WITH 6 FOOT O.X. [IN]
1 Foot of Compacted Fill	0.9	1.1	0.5	0.3	0.2
2 Feet of Compacted Fill	1.7	2.2	1.2	0.8	0.5
3 Feet of Compacted Fill	2.4	3.1	1.9	1.2	0.8
4 Feet of Compacted Fill	3.0	3.9	2.5	1.7	1.1

COLUMN FOOTING LOADS[2]

LOAD CONDITION	MINIMUM SETTLEMENT [IN]	MAXIMUM SETTLEMENT [IN]	MAX. SETTLEMENT WITH 2 FOOT O.X. [IN]	MAX. SETTLEMENT WITH 4 FOOT O.X. [IN]	MAX. SETTLEMENT WITH 6 FOOT O.X. [IN]
20 KIPS	1.4	2.6	1.1	0.3	0.1
50 KIPS	1.8	3.3	1.8	0.7	0.2
100 KIPS	2.2	3.9	2.3	1.2	0.4
200 KIPS	2.5	4.4	2.9	1.6	0.6

CONTINUOUS FOOTING LOADS[3]

LOAD CONDITION	MINIMUM SETTLEMENT [IN]	MAXIMUM SETTLEMENT [IN]	MAX. SETTLEMENT WITH 2 FOOT O.X. [IN]	MAX. SETTLEMENT WITH 4 FOOT O.X. [IN]	MAX. SETTLEMENT WITH 6 FOOT O.X. [IN]
1,500 PSF	0.9	1.2	0.6	0.2	0.1
2,000 PSF	1.1	2.2	0.8	0.3	0.1

NOTES:

1) Surcharge fill settlements are based on the assumption that the fill is uniformly distributed over a circular area with a 100 foot radius.

3) Continuous footing settlements assume that the footing has a minimum width of 12 inches.

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FOUNDATION SETTLEMENT ESTIMATES

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FIGURE 8

C-5.4). The laboratory test results were used to develop settlement models corresponding to the interpreted soil profiles from the CPT soundings. The "minimum" and "maximum" settlements in the table reflect the variation in settlement estimated from the different interpreted CPT soil profiles. The settlement estimates are based on limited laboratory data. There is not sufficient laboratory data currently available to conduct a statistical analysis of the range of settlement possible due to variable soil conditions at the site. The actual settlements resulting from the proposed foundation loads may vary considerably from the values presented in Figure 8.

For each loading condition, the estimated reduction in settlement resulting from the over-excavation of the surficial materials is also presented in Figure 8. Note that the over-excavation depths reflect depths below the application of the load. For surcharge fill loads, over-excavation depths would reflect existing ground surface elevations. For foundation loads, the over-excavation depths reflect the depth below the bottom of the foundations (2 feet below existing ground surface elevations).

The values in Figure 8 may be used to give a preliminary indication of the approximate remedial grading and surcharge loading that will be necessary at the site. For example, if a lightly loaded structure will use continuous footings loaded at 1,500 PSF, we estimate that the total settlement will vary from about 0.9 to 1.2 inches. If a total settlement of about ½ inch is desired for this structure, then over-excavating 2 feet below the footing bottoms (4 feet below finish grade) may be sufficient (a total settlement of about 0.6 inches is shown in the table). As another example, if a minor structure has a 100 kip column load which bears at 2 feet below grade with an allowable pressure of 1,500 PSF, we estimate that the total settlement would vary from about 2.2 to 3.9 inches. If a total settlement of less than ½ inch is desired, the area could be surcharged with 4 feet of compacted fill for 2 to 4 months as described in Section 8.3.3. The surcharge may reduce the anticipated foundation settlement by between 3.0 and 3.9 inches. Alternatively, the soil under the structure could be over-excavated and compacted to a depth of 6 feet below foundation bottoms (or 8 feet below finish grade). This may reduce the total anticipated settlement to about 0.4 inches.

8.4.2 Deep Foundations : Driven piles should be used to support the major structures, as described in Section 8.3.5. Prior to driving the piles, ground improvement should be conducted in order to mitigate the liquefaction hazard and associated post-liquefaction settlement. Additional subsurface investigation should be conducted after the ground improvement is completed in order to confirm that the liquefaction hazard has been mitigated.

Piles should be embedded into the dense sand deposits at approximately 35 to 45 feet below the existing ground surface. Pile embedment depth should be confirmed in the field based on the results of indicator piles and load tests. The indicator piles should be placed near the CPT sounding locations in order to improve required pile length estimates and establish driving criteria. We recommend driving at least 8 indicator piles to depths where the required axial capacity is attained based on pile driving analysis. Indicator piles may be used as production piles if they are driven into their planned location. After the indicator piles are installed, at least two of the piles should be tested to confirm the load carrying capacity. Both downward and uplift capacities should be verified. The pile load tests should be conducted in general accordance with ASTM D1143. Any necessary pile modifications should be made after the indicator piles are load tested.

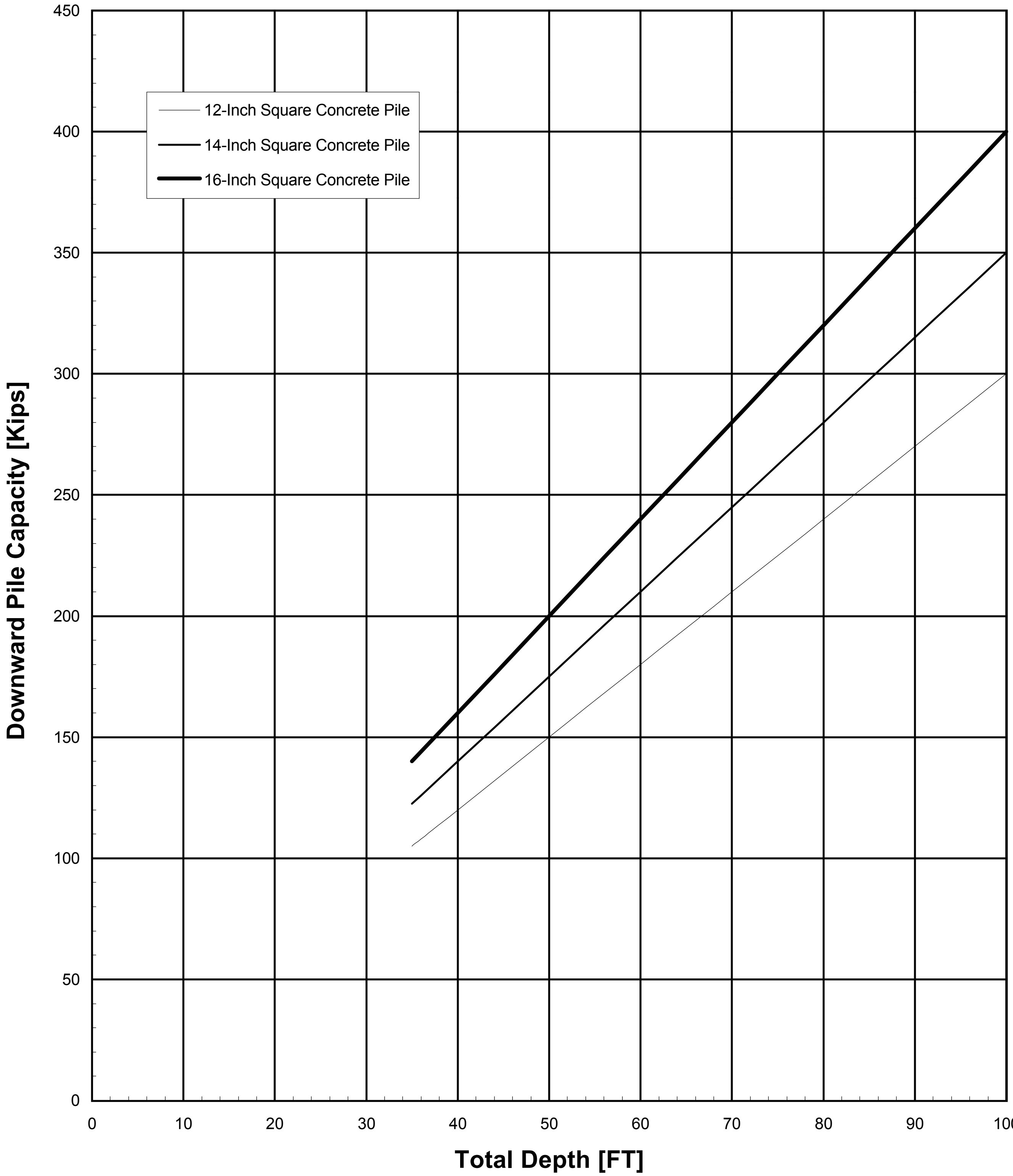
8.4.2.1 Axial Pile Capacity (Downward) : The allowable axial capacities of 12, 14 and 16 inch square, precast concrete piles are presented in Figure 9. These capacities were developed using the results of the CPT soundings, and the prediction algorithms of the French LCPC method (Bustamante and Gianeselli, 1982). The results of a blind pile capacity prediction study suggested that the LCPC prediction method was "...the best method with a maximum error of about 25 percent, an average error of 0 percent, and a standard deviation of 15 percent" (Robertson et al, 1988). The LCPC analyses for each CPT sounding are presented in detail in Appendix G.

The allowable capacities given in Figure 9 represent the lower bound of the predictions made by the LCPC method. Assuming that the piles are terminated in the dense sands as recommended, the actual pile capacities should be 2 to 3 times the allowable capacities presented in Figure 9. The pile load tests will be used to confirm the actual safety factor. Piles in groups should be spaced at least 2½ pile diameters on center (but not less than 3 feet). No reduction in downward pile capacities due to group action will be necessary, assuming that the piles are adequately spaced. Note that a one-third increase for short-term wind or seismic loads may be used. Note also that the allowable axial pile capacities shown in Figure 9 may need to be reduced to reflect the structural capacity of the piles. The allowable downward axial pile capacity based on soil considerations alone may also be determined from the following equation:

$$Q_{all} = 3 * z * (d/12)$$

where: Q_{all} = Allowable Downward Capacity [Kips]
z = Total depth of pile below ground surface [FT]
d = Pile dimension (12, 14 or 16) [IN]

8.4.2.2 Axial Pile Capacity (Uplift) : Uplift forces for driven piles may be resisted by friction developed along the perimeter of the pile, as well as by the weight of the pile itself. The allowable uplift capacity for piles embedded



Notes: 1) These curves incorporate an estimated safety factor of 2 to 3 (to be verified by pile load tests).
2) These pile capacities may need to be reduced based on structural considerations.

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DOWNWARD PILE CAPACITY

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FIGURE 9

into the on site soils is given in Figure 10. Note that the allowable capacities assume that the ground around the piles has been improved to mitigate liquefaction. The following equation may be used to calculate uplift capacity:

$$T_{all} = T + W$$

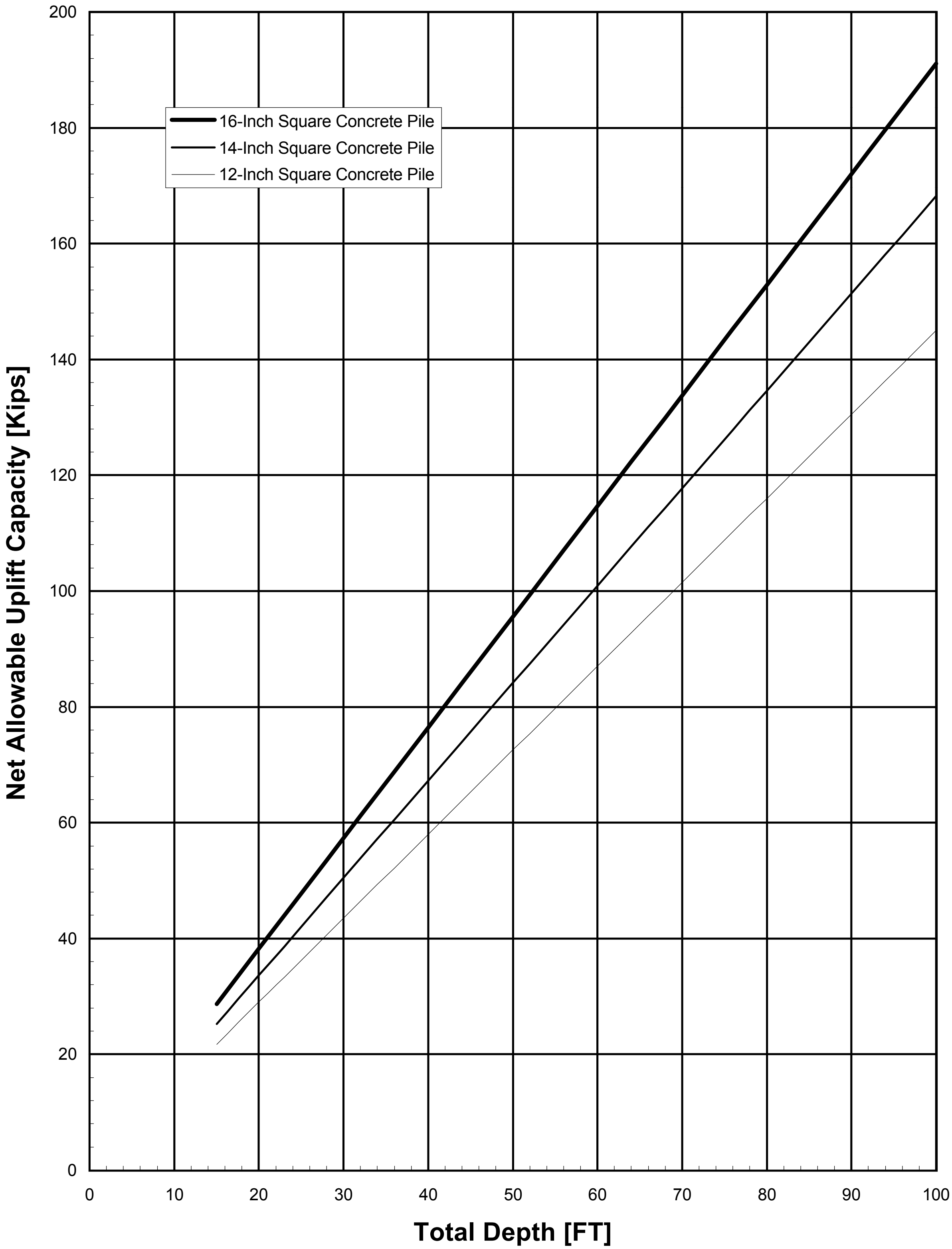
where: T_{all} = Allowable Uplift Capacity [Kips]
T = Uplift capacity from Figure 10
W = Weight of the pile [Kips]

The allowable uplift capacities presented in Figure 10 were estimated from the downward capacities presented in Figure 9. The lower bound of the downward capacity was assumed to approximate the frictional contribution of the site soils (the safety factor in the downward capacities will be developed from end bearing in the dense sands). The lower bound of the downward capacities were reduced slightly to reflect probable end bearing effects in the saturated clays. A factor of safety of 2 was then applied to the modified downward capacities to develop the allowable uplift capacities presented in Figure 10.

According to Archimedes' principle, the proposed concrete lined brine ponds will experience a upward bouyant force equal to the weight of the fluid that will be displaced. These ponds will therefore need to be anchored to the ground using piles designed primarily for uplift. The values presented in Figure 10 are appropriate for friction piles embedded at least 15 feet into the ground with a minimum spacing of 10 feet, center to center. Piles that are spaced closer than 10 feet will likely experience a reduction in the uplift capacity due to group effects. Additional uplift capacity recommendations may be developed based on the actual pile configurations.

8.4.2.3 Lateral Pile Capacity : Lateral loads on structures with deep foundations may be resisted by the passive soil pressures developed against the pile caps. A passive pressure of 300 lb/ft^3 is recommended. In addition to the passive pressures, the flexural strength of the piles will also resist lateral forces. Consequently, a simplified lateral pile analysis was conducted using Winkler's model (an idealized beam on an elastic foundation), assuming unimproved soil conditions. Note that ground improvement will significantly increase the lateral rigidity of the site soils.

The results of the simplified lateral analyses for a pile with a minimum width of 12 inches is presented in Figures 11a and 11b. According to Figure 11a, the lateral load at the pile cap required to produce ¼ inch of lateral deflection is approximately 2 kips. According to Figure 11b, the applied moment at the pile cap required to produce ¼ inch deflection is 14 foot-kips. Note that the deflections may be scaled by the actual loads, and that the solutions may be superposed to estimate deflections from other applied shears and moments.



Notes: 1) These curves incorporate an estimated safety factor of 2 (to be verified by pile load tests).
2) These pile capacities may need to be reduced based on structural considerations.

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PILE UPLIFT CAPACITY

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FIGURE 10

The characteristic shape of the solutions shown in Figures 11a and 11b assumes uniform soil conditions with a free pile cap. The actual deflection, shear, and moment response will vary based on the actual loading conditions as well as variability in the supporting soil strata at depth. Once the lateral design loads for the piles are better known, more sophisticated lateral load analyses could be conducted using the program LPILE.

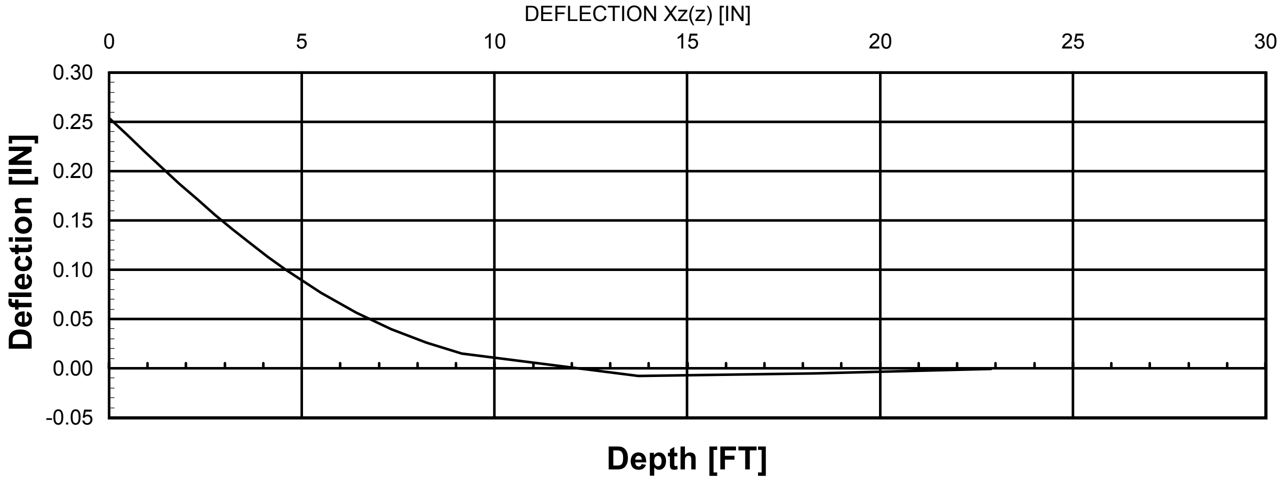
8.4.2.4 Settlement: Total and differential settlements of approximately ½ inch or less are estimated for any of the proposed structures that will be supported by driven piles. This estimate assumes that the pile tips will bear in the dense sand layers, and that the ground will be improved to mitigate liquefaction so that the piles will not be subjected to downdrag. Settlement estimates for the deep foundations should be confirmed by pile load tests.

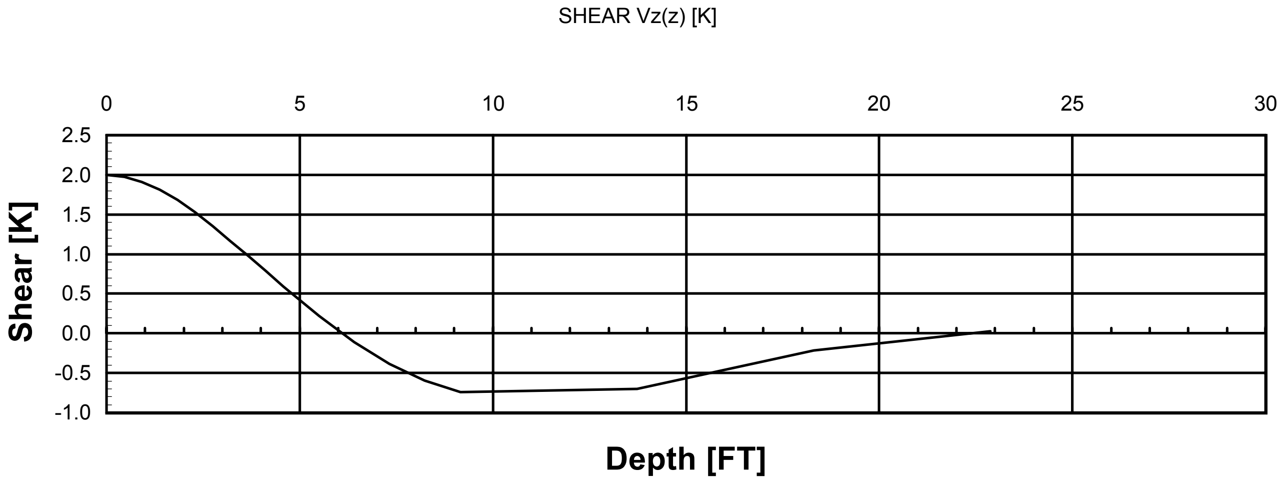
8.4.2.5 Installation: All piles should be driven under the observation of Geotechnics Incorporated. Piles should be driven to the predetermined design lengths, unless adjusted based on the results of the indicator piles and load tests. Due to the anticipated difficulty in driving through the upper sand beds, we recommend that the piles be pre-drilled to a maximum depth of 10 feet above the design pile tip elevation. The area of the pre-drilled hole should not exceed 75 percent of the cross-sectional area of the pile.

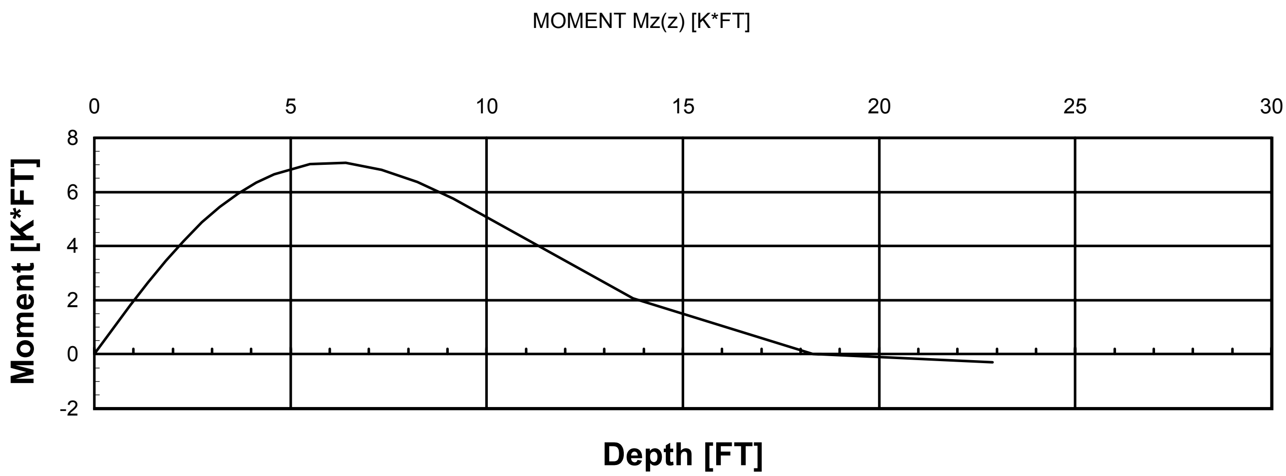
8.5 Seismic Design

The proposed geothermal plant will be underlain by several thousand feet of soil. Shear wave velocity measurements were conducted at five foot intervals for CPT soundings 2, 3 and 5 up to a total depth of 100 feet below grade. The results are presented in the figures of Appendix B. The shear wave velocity measurements in the upper 40 feet experienced interference, possibly due to equipment vibrations from a nearby geothermal plant. However, the average shear wave velocity in the soils between 40 and 100 feet was approximately 600 ft/s (the velocities varied from about 435 to 1500 ft/s).

Ground improvement will be conducted under all settlement sensitive structures in order to mitigate the liquefaction hazard. Ground improvement may increase the average shear wave velocities at the site to greater than 600 ft/s. The actual increase in shear wave velocity due to ground improvement is not known. Additional subsurface investigation could be used after ground improvement to estimate the increase in shear wave velocity. However, for design purposes, we recommend using a conservative shear wave velocity (Vs) of 600 ft/s. According to the 1997 UBC criteria, an average shear wave velocity of 600 ft/s or greater would suggest that the site may be classified as seismic soil profile S_D (a deep soil site).







PIER DIAMETER:	1.0	FT
LATERAL LOAD:	2.0	KIP
APPLIED MOMENT:	0.0	K*FT

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LATERAL PILE LOADS

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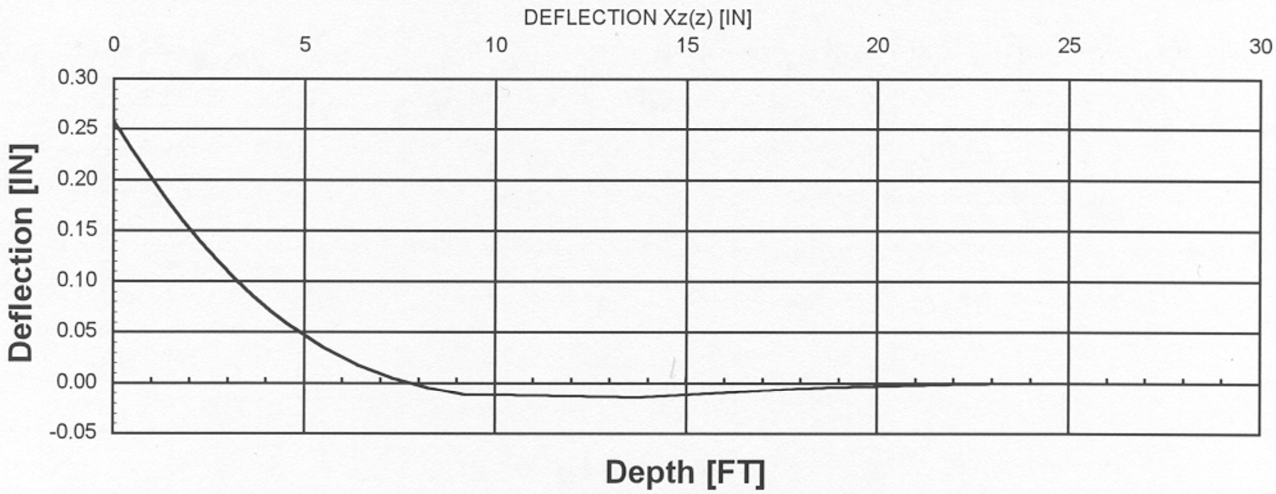
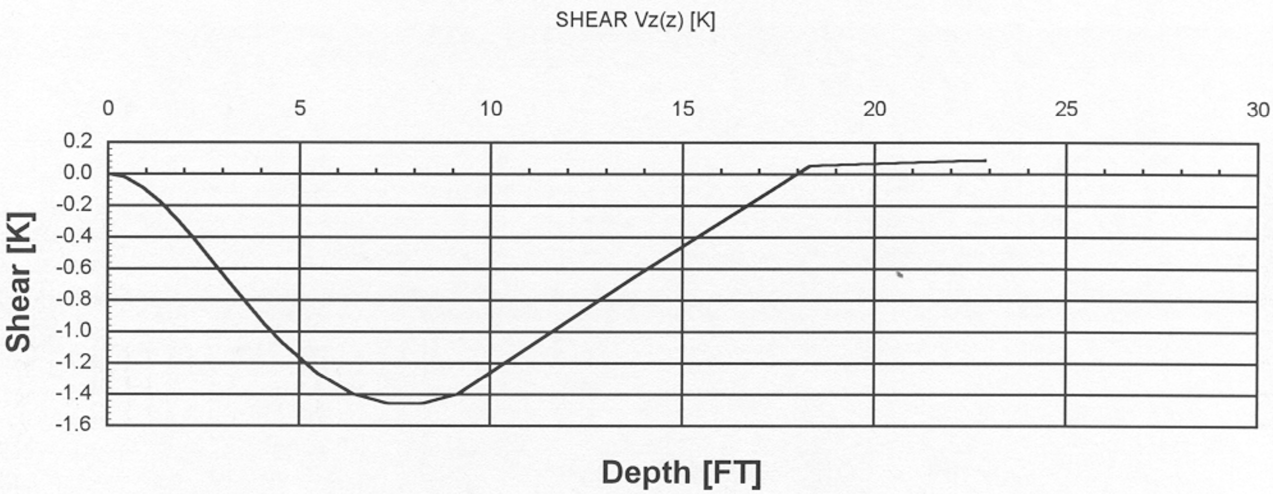
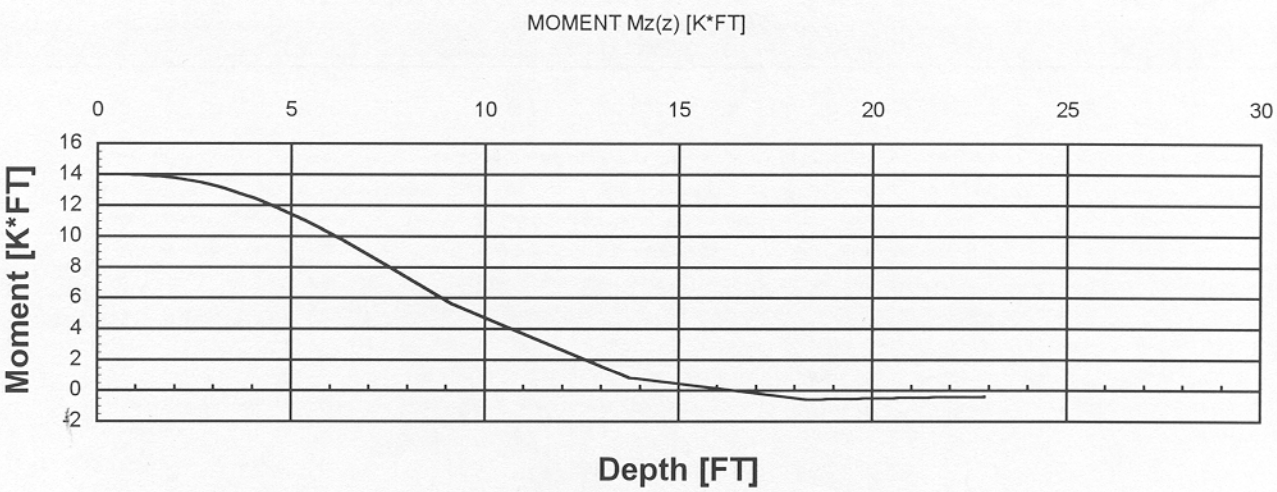
FIGURE 11a







PIER DIAMETER:	1.0	FT
LATERAL LOAD:	0.0	KIP
APPLIED MOMENT:	14.0	K*FT

For saturated fine grained soils subjected to rapid loading, Poisson's ratio (ν) is commonly assumed to be 0.5 (incompressible). Laboratory studies have shown that Poisson's ratio for soft saturated soils typically varies between 0.45 and 0.5 (Ishihara, 1996). The general relationship between primary or compression wave velocity (Vp) and shear wave velocity (Vs) may be given as:

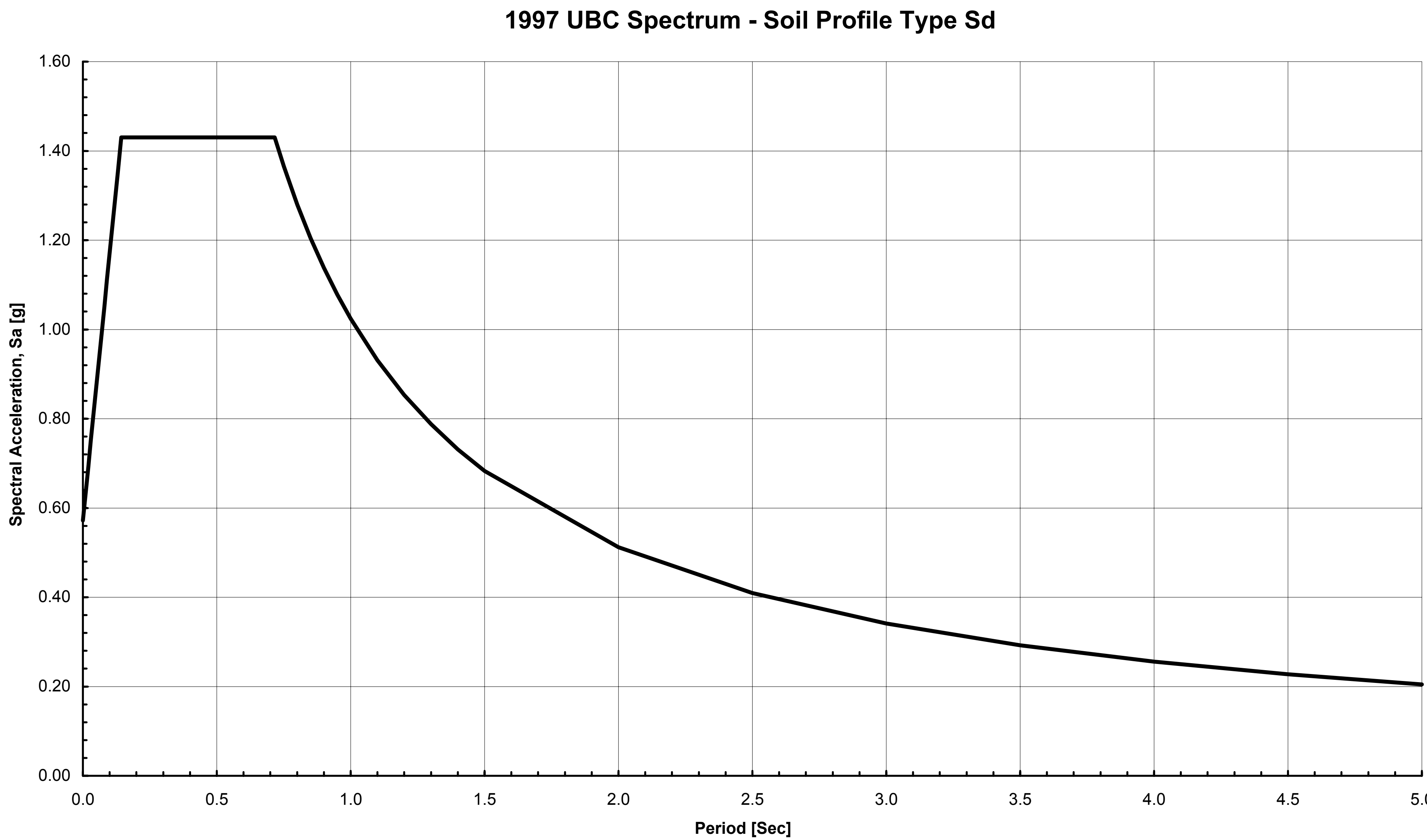
$$Vp = Vs * [(2-2\nu)/(1-2\nu)]^{1/2}$$

Seismologists often assume that general geologic materials at depth have an average Poisson's ration of roughly 0.3. For a Poisson's ratio of between 0.3 and 0.45, the above equation indicates that the primary wave velocity will be roughly 1.9 to 3.3 times the shear wave velocity (roughly 1,000 to 2,000 ft/s). Note that as Poisson's ratio approaches 0.5, the compression wave velocity approaches infinity.

8.5.1 1997 UBC Seismic Parameters: Based on the 1997 Uniform Building Code (UBC) seismic hazard maps, the subject site is situated within 2 km of the Brawley Fault (we have referred to this as the Brawley Seismic Zone in Section 6.1). The Brawley Fault is a Type B Seismic Source based on the 1997 UBC criteria. The subject site is situated in 1997 UBC Seismic Zone 4 (Z = 0.40). Since the distance between the site and the nearest active fault is less than 2 km, the 1997 UBC near source acceleration and velocity factors (N_a and N_v) equal 1.3 and 1.6, respectively. Based on the shear wave velocity measurements described above, and assuming that the liquefaction potential is mitigated in the building areas in general accordance with our recommendations, it is our opinion that a 1997 UBC seismic Soil Profile S_D may be applied to the site. The seismic acceleration and velocity coefficients (C_a and C_v) would equal 0.57 and 1.02, respectively. The resulting 1997 UBC response spectra is presented in Figure 12a. Design of structures should comply with the requirements of the governing jurisdictions, building codes and practices of the Association of Structural Engineers of California.

8.5.2 Site Specific Response Spectra: Probabilistic seismic hazard analysis was used to develop a uniform hazard response spectrum for the subject site. As a preliminary check on the fault model parameters used for the FRISKSP analysis, the USGS uniform hazard spectra for the site with a 475-year return period (10 percent probability of exceedance in 50 years) was developed, as shown in Figure 12b. The spectral ordinates for peak ground acceleration (PGA) as well as periods of 0.2, 0.3 and 1.0 second were obtained from the USGS website by entering the site latitude and longitude (http://geohazards.cr.usgs.gov/eq/). Note that the USGS response spectrum is applicable only for rock sites. It is interesting to note that the USGS peak ground accelerations for a rock site located at the subject site coordinates would be 1.17g, as shown in Figure 12b.

The 475-year return period uniform hazard spectrum developed for the site using the program FRISKSP is presented in Figure 12c, along with the same USGS spectrum



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SPECTRAL ACCELERATION

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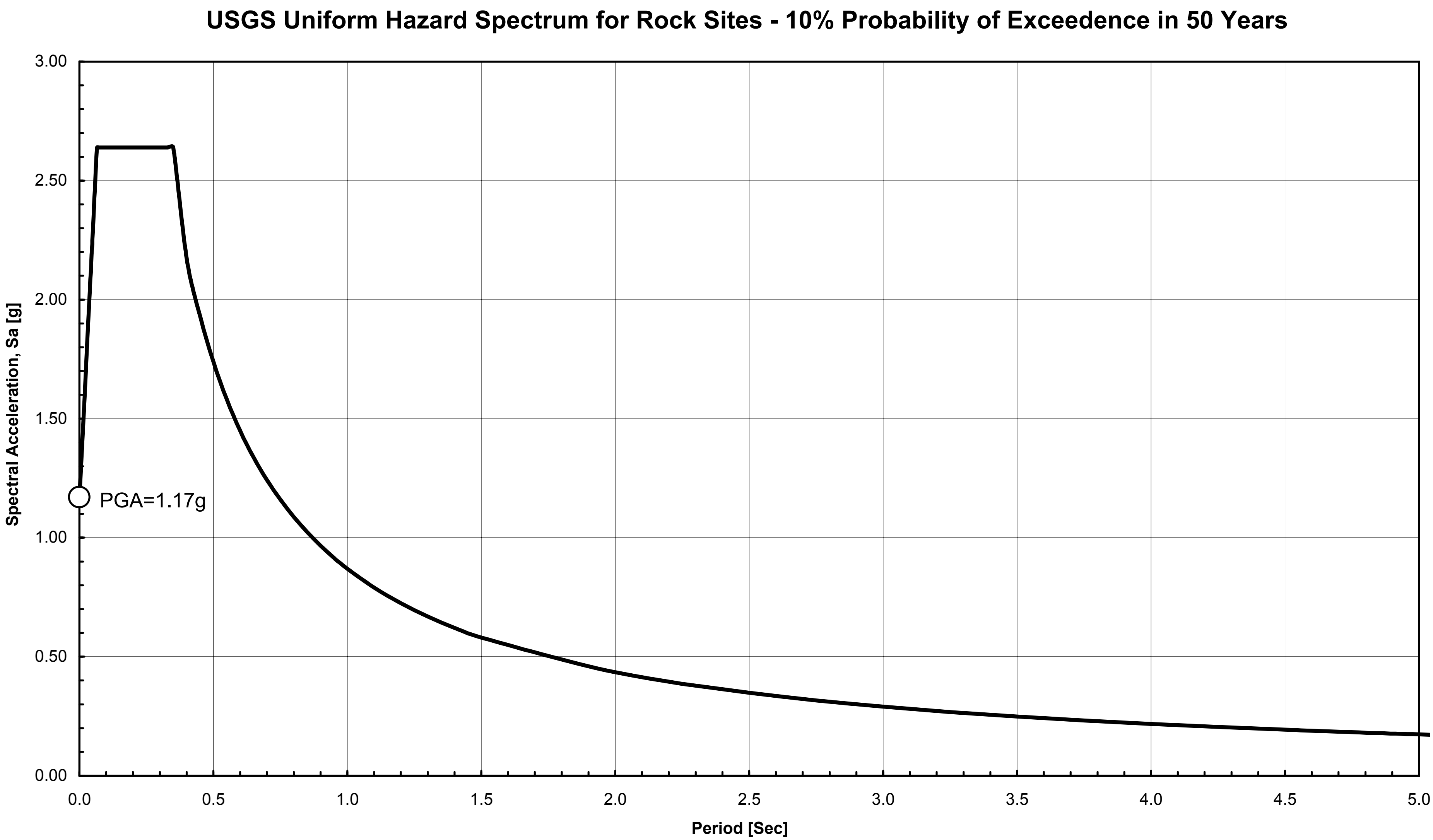
FIGURE 12a



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SPECTRAL ACCELERATION

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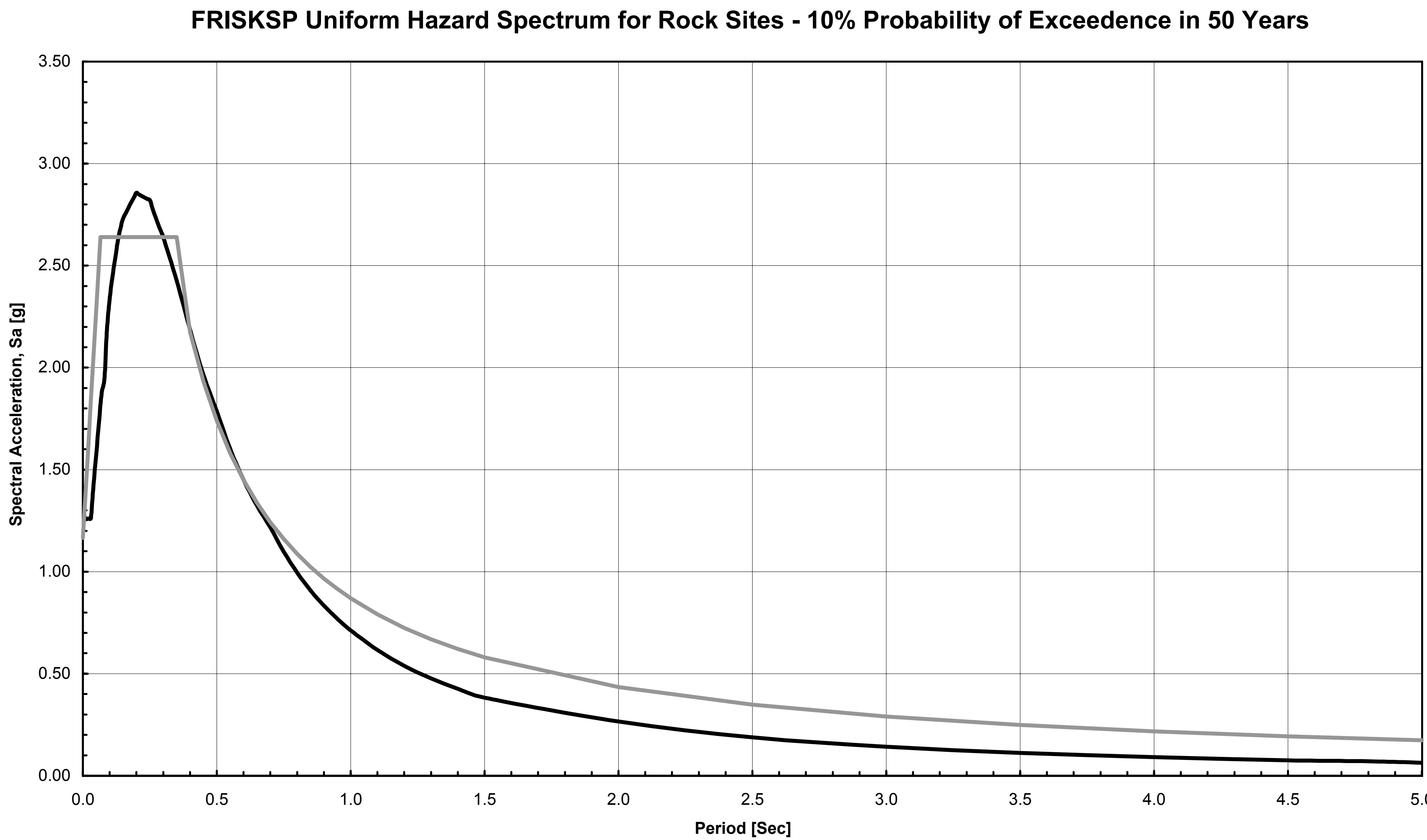
Document No. 02-0022

FIGURE 12b



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SPECTRAL ACCELERATION

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Document No. 02-0022
FIGURE 12c

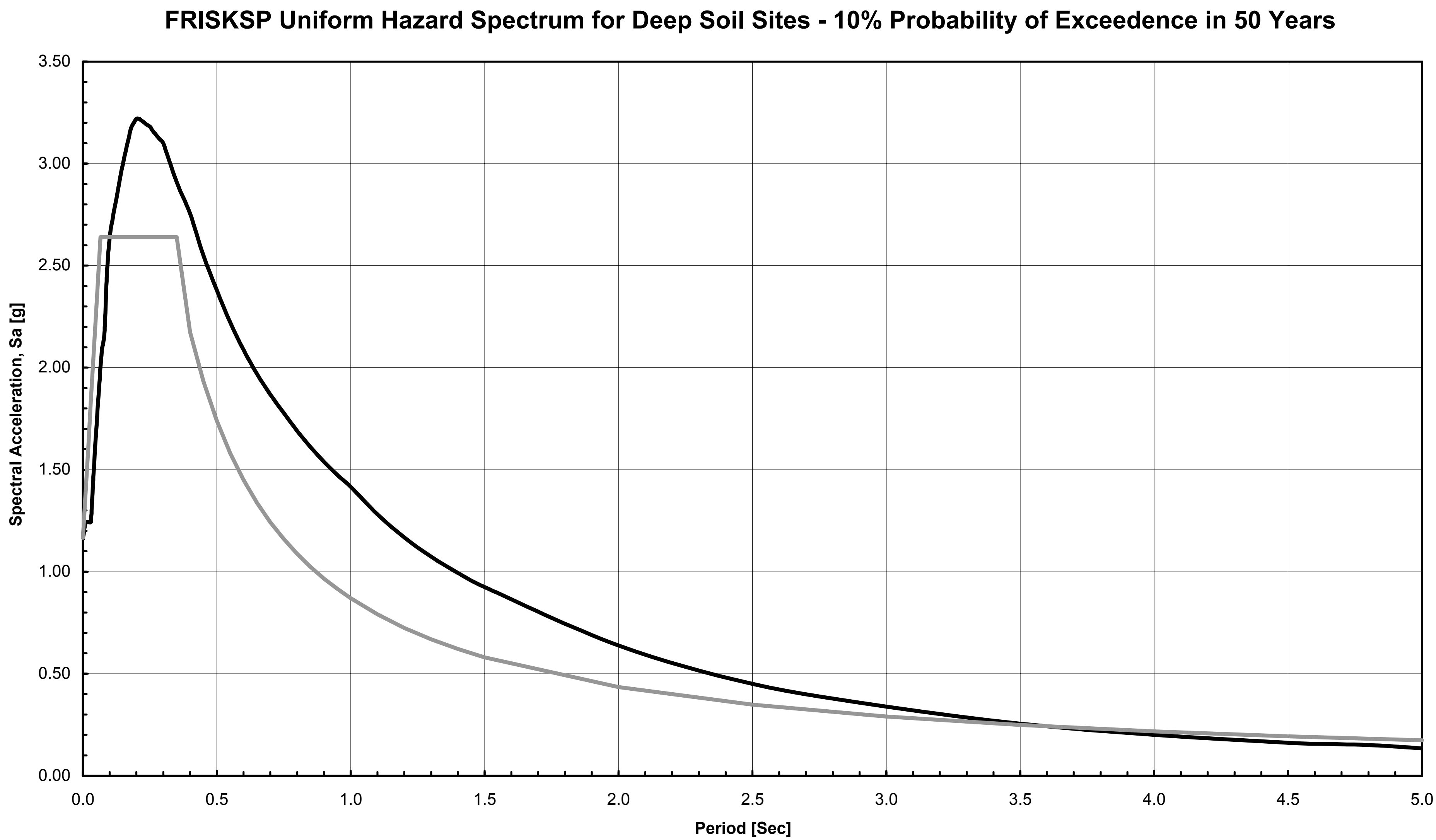

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FIGURE 12d

presented in Figure 12b for comparison purposes. Once again, the FRISKSP spectrum was developed assuming rock site conditions. It is our opinion that the FRISKSP spectrum correlates reasonably well with the generally accepted USGS spectrum for the site. It should be noted that some variability should be expected between the two spectra because different attenuation relationships were used, and because the USGS spectrum is only based on the four spectral ordinates noted above.

In order to develop a site specific response spectrum for design of structures at the site, the program FRISKSP was again used. However, the attenuation relationship was changed to one developed specifically for data taken from previous earthquakes at deep soil sites (Sadigh, 1997). Considering the high levels of uncertainty in the composition and character of the deep soil deposits which underlie the site, it is our opinion that a probabilistically developed site specific response spectrum is more meaningful than one determined from a computational analysis such as SHAKE with a poorly defined deep soil profile. The resulting site specific response spectrum is presented in Figure 12d. Note that all of the spectral ordinates in Figure 12d are estimated to have a 10 percent probability of being exceeded in a 50 year period.

8.6 On-Grade Slabs

Building slabs should be supported by compacted fill as recommended in Section 8.3. Slabs should be designed for the anticipated loading, using soil parameters which reflect the as-graded subgrade conditions. For slabs constructed on compacted site soils, a modulus of subgrade reaction of 100 lbs/in^3 may be used. Such slabs should be at least 6 inches thick, and should be reinforced with at least No. 3 bars on 18-inch centers, each way.

The subgrade modulus could be improved if the upper soils were compacted after ground improvement with stone columns. For slabs founded on a 2 to 4 foot thick compacted fill mat composed of the on site soils mixed with gravel, a modulus of subgrade reaction of 150 lbs/in^3 or more may be appropriate.

8.6.1 Moisture Protection for Slabs: Concrete slabs constructed on soil ultimately cause the moisture content to rise in the underlying soil. This results from continued capillary rise and the termination of normal evapotranspiration. Because normal concrete is permeable, the moisture will eventually penetrate the slab. Excessive moisture may cause mildewed carpets, lifting or discoloration of floor tile, or similar problems. The amount of moisture transmitted through the slab can be controlled by the use of various moisture barriers.

To decrease the likelihood of problems related to damp slabs, suitable moisture protection measures should be used where moisture sensitive floor coverings or other factors warrant. The most commonly used moisture protection in southern California consists of about two inches of clean sand covered by 'visqueen' plastic sheeting. In addition, two inches of sand are placed over the plastic to decrease concrete curing problems associated with placing concrete directly on an impermeable membrane. It has been our experience that such systems will transmit from approximately 6 to 12 pounds of moisture per 1000 square feet per day. The project architect should review these estimated transmission rates, since these values may be excessive for some applications and may exceed flooring manufacturers warranties. If more protection is needed, Geotechnics Incorporated should be contacted for additional recommendations. Other alternatives may include the use of open graded rock or pea gravel, and/or a warranted water proofing membrane in addition to concrete with a low water to cement ratio.

8.6.2 Exterior Slabs: Exterior slabs and sidewalks should be at least 4 inches thick, and should be reinforced with at least 6x6 W2.9/W2.9 welded wire fabric (WWF) placed securely at mid-height of the slabs. Crack control joints should be placed on a maximum of 10 foot centers, each way, for slabs, and 5 foot centers for sidewalks.

8.6.3 Reactive Soils: In order to aid in evaluating the corrosion potential for buried metal improvements, selected samples were tested for pH and resistivity in general accordance with CTM 643 (see Figure C-2). Based on the test results, the on-site soils appear to be severely corrosive to ferrous metals. A corrosion consultant should be contacted to provide specific corrosion control recommendations.

In order to assess the sulfate exposure of concrete in contact with the site soils, samples were tested for water soluble sulfate content. The test results are also reported in Figure C-2. Based on these test results, the site soils appear to have a moderate potential for sulfate attack based on UBC criteria. The project design engineer may choose to use the sulfate test results in conjunction with Table 19-A-4 of the 1997 UBC in order to specify a suitable cement type, water cement ratio, and minimum compressive strength for concrete used on site which will be in direct contact with soil, including all foundations and slabs. Studies have shown that the use of improved cements in the concrete, and a low water-cement ratio will improve the resistance of the concrete to sulfate exposure.

8.6.4 Expansive Soils: Laboratory testing indicates that the surficial clays at the site have a medium expansion potential based on UBC criteria. The test results are presented in Figure C-3. In order to reduce the potential for distress to structures with slabs-on-grade, we have recommended a minimum slab thickness of 6 inches, with at least No. 3 bars on 18-inch centers, each way. For exterior slabs and sidewalks, we have recommended the use of reinforcement to reduce the magnitude of differential movement across the crack control joints.

8.7 Earth-Retaining Structures

Backfilling retaining walls with expansive soil can increase lateral pressures well beyond normal active or at-rest pressures. We recommend that earth retaining structures be backfilled with soil having an expansive index of 20 or less. The backfill area should include the zone defined by a 1:1 sloping plane, extending back from the heel of the wall. Any surcharge loads placed within the backfill area should be accounted for in the structural design. Backfill should be compacted to at least 90 percent relative compaction, based on ASTM D1557. Backfill should not be placed until walls have achieved adequate structural strength. Heavy compaction equipment which could distress the walls should not be used.

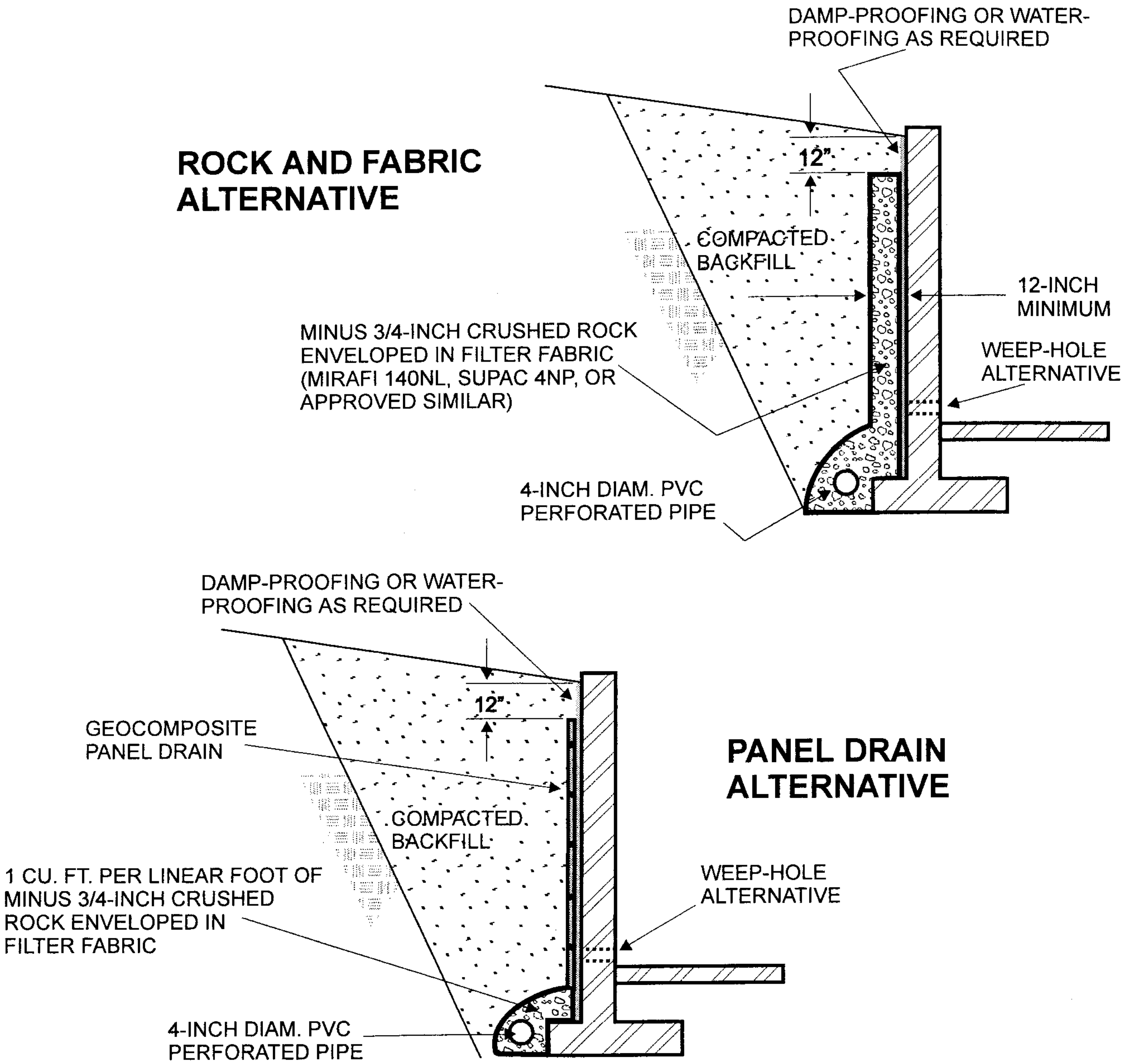
8.7.1 Retaining Walls : Cantilever retaining walls with level backfill should be designed for an active earth pressure approximated by an equivalent fluid pressure of 35 lbs/ft^3. This assumes that the walls are backfilled with freely draining select granular materials. The active pressures should be used for walls free to yield at the top at least 0.2 percent of the wall height. The above pressures do not consider surcharge loads or hydrostatic pressures. Walls should contain an subdrain to eliminate any hydrostatic forces. The recommended wall drain details are presented in Figure 13.

8.7.2 Basements : Any subgrade walls constructed at the site will likely be subjected to groundwater forces. This will depend upon the final grade of the site and basement depth, and should be evaluated during the plan review process. Although a permanent dewatering system may be an alternative, such systems need to be rigorously maintained in order to avoid hydrostatic pressure buildup and wall failure, and may require a groundwater discharge permit. Consequently, we recommend that the basement walls be water-proofed, and designed to withstand the anticipated hydrostatic loads. Laterally restrained basement walls below the groundwater level should be designed using an equivalent fluid pressure of 95 lbs/ft^3. The uplift force on the basement slab should also be accounted for in structural design.

8.8 Preliminary Pavements

Alternative recommendations are provided below for using either asphalt concrete or Portland cement concrete (PCC) pavements at the subject site. These preliminary recommendations are based on the result of a single R-Value test conducted on surficial soils collected during the subsurface investigation (Figure C-6). These recommendations should be considered subject to revision based on the results of additional R-Value testing conducted on representative samples of the final pavement subgrade materials.

In general, the upper 12 inches of pavement subgrade should be scarified, brought to about optimum moisture content, and compacted to at least 95 percent of maximum dry density as determined by ASTM D1557. Subgrade preparation should be conducted immediately prior to the placement of the pavement section. All aggregate base should also be compacted to at least 95 percent of the maximum density as determined by ASTM D1557. Asphalt concrete should be compacted to at least 95 percent of the Hveem density (ASTM D1562).



NOTES

1) Perforated pipe should outlet through a solid pipe to a free gravity outfall. Perforated pipe and outlet pipe should have a fall of at least 1%.
2) As an alternative to the perforated pipe and outlet, weep-holes may be constructed. Weep-holes should be at least 2 inches in diameter, spaced no greater than 8 feet, and be located just above grade at the bottom of wall.
3) Filter fabric should consist of Mirafi 140N, Supac 5NP, Amoco 4599, or similar approved fabric. Filter fabric should be overlapped at least 6-inches.
4) Geocomposite panel drain should consist of Miradrain 6000, J-DRain 400, Supac DS-15, or approved similar product.
5) Drain installation should be observed by the geotechnical consultant prior to backfilling.

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WALL DRAIN DETAILS

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FIGURE 13

8.8.1 Asphalt Concrete: Four traffic types were assumed for design: areas of light traffic and passenger car parking (Traffic Index of 4.5 to 5.0), and access drives and truck routes (Traffic Index of 6.0 to 7.0). The project civil engineer should review these values to determine which are appropriate. Laboratory R-Value tests indicate that an R-Value of 7 may be used for preliminary design. Based on the assumed Traffic Indices and R-Value, the following pavement sections are recommended in accordance with the CALTRANS design method.

TRAFFIC INDEX	ASPHALT CONCRETE	AGGREGATE BASE
4.5	3 inches	8 inches
5.0	3 inches	10 inches
6.0	4 inches	11 inches
7.0	4 inches	15 inches

Asphalt concrete should conform to *Standard Specifications for Public Works Construction (SSPWC)*, Section 400-4. Aggregate base should conform to *SSPWC* Section 200-2 for crushed aggregate or crushed miscellaneous base, or to Section 39 of the Caltrans Standard Specifications for Class 2 base.

8.8.2 Portland Cement Concrete: Concrete pavement design was conducted in accordance with the simplified design procedure of the Portland Cement Association. This methodology is based on a 20 year design life. For design, it was assumed that aggregate interlock joints will be used for load transfer across control joints. Furthermore, the Portland cement concrete was assumed to have a minimum 28 day flexural strength of 600 psi. Laboratory R-Value tests conducted on a representative sample of the subgrade soils indicate that these materials will provide “low” subgrade support (corresponding to a modulus of subgrade reaction less than 120 pci). Based on these assumptions, we recommend that the pavement section generally consist of 6 inches of Portland cement concrete over native subgrade. Heavy truck traffic areas should consist of 7 inches of Portland cement concrete over subgrade. Crack control joints should be constructed for all PCC pavements on a maximum spacing of 10 feet, each way. Concentrated truck traffic areas, such as loading docks, should be reinforced with at least number 4 bars on 18-inch centers, each way.

9.0 LIMITATIONS OF INVESTIGATION

This investigation was performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional opinions included in this report. The samples taken and used for testing and the observations made are believed representative of the project site. However, soil and geologic conditions can vary significantly between borings. If this occurs, the changed conditions must be evaluated by the geotechnical consultant and additional recommendations made, if warranted.

This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the necessary design consultants for the project and incorporated into the plans, and the necessary steps are taken to see that the contractors carry out such recommendations in the field.

The findings of this report are valid as of the present date. However, changes in the condition of a property can occur with the passage of time, whether due to natural processes or the work of man on this or adjacent properties. In addition, changes in applicable or appropriate standards of practice may occur from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

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APPENDIX A

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APPENDIX B

SUBSURFACE EXPLORATION

Field exploration consisted of a visual reconnaissance of the site, the advancement of cone penetrometer soundings (CPT), and the drilling of exploratory borings. The subsurface investigation was performed on November 29 and 30, 2001. A 20 ton capacity rig was used to perform nine CPT soundings at the site. CPT soundings were advanced to a maximum depth of 100 feet. An 8-inch diameter, truck mounted hollow stem drill rig was used to conduct the exploratory borings. Water was used to support the excavations, and reduce the potential for heaving at sampling locations. Two borings were drilled at the site to a maximum depth of 77½ feet. The approximate locations of the CPT soundings and hollow stem borings are shown on the Site Plan, Figure 2. The CPT sounding and boring logs are presented in the following figures.

The nine electronic cone penetration soundings (CPT) were conducted by Gregg In-Situ. The layout of the cone penetrometer tip and data acquisition system, as well as the soil behavior and pore pressure interpretations are presented in the following figures. The cone penetrometer test consists of advancing a cone-tipped cylindrical probe into the ground using hydraulic down pressure. While the probe is being advanced, the resulting resistance to penetration is monitored continuously by a computer. The probe contains two strain gauge load cells which measure the soil bearing resistance acting on the conical tip, as well as the frictional resistance along the sleeve. These two readings are recorded continuously with depth, and transmitted as electronic signals to a data acquisition system mounted inside the rig. The raw strain gage data from the CPT soundings is shown in Figures B-1 through B-9. Associated soil behavior interpretations, pore pressure dissipations, and shear wave velocity measurements from each sounding are presented between Figures B-1 through B-9.

The CPT soundings were performed in general accordance with ASTM D5778. A 20 ton capacity rig was used for all of the soundings. The computer recorded tip resistance (q_c), sleeve friction (f_s), and dynamic pore pressure (U) at 5 cm depth intervals. The pore water pressure element was located behind the cone tip, and consisted of a porous medium which was saturated prior to penetration. Transient dynamic pore pressures were recorded during cone advancement, and aided in defining soil behavior types. Static pore water pressures were determined by stopping the cone below the groundwater table and waiting for the pore pressures to stabilize. Pore pressure dissipations were recorded at 5 second intervals during pauses in the penetration. Selected pore pressure dissipation curves are included for CPT soundings 1, 4, 6, 7, 8 and 9.

APPENDIX B

SUBSURFACE EXPLORATION (Continued)

On CPT soundings 2, 3 and 5 the probe was stopped at five foot intervals, and the ground surface was excited with a 10 pound sledge hammer blow to a steel beam. An accelerometer in the CPT tip was then used to measure shear wave arrival times. The raw shear wave velocity data measurements, and the interpreted shear wave velocity profiles are presented for CPT soundings 2, 3 and 5. According to the operators for Gregg In-Situ, shear wave data from depths of less than 40 feet was distorted in all three of these soundings. The operators hypothesize that the distortions may have been the result of nearby machinery vibrations.

Two hollow stem borings were conducted at the site to collect samples for laboratory testing and provide supplemental soil classification data. The hollow stem boring logs are presented in Figures B-10 through B-14. Disturbed samples were collected from the exploratory borings using a Standard Penetration Test (SPT) sampler (2-inch outside diameter). SPT samples were sealed in plastic bags, labeled, and returned to the laboratory for testing. The drive weight for the SPT samples was a 140-pound hammer with a free fall of 30 inches. For each sample, we recorded the number of blows needed to drive the sampler 6, 12, and 18 inches. The number of blows needed to drive the final 12 inches is shown on the attached logs under "blows per ft". Undisturbed samples were also collected using 3-inch diameter, 36-inch long, thin walled sample tubes (Shelby Tubes). The Shelby Tubes were driven with hydraulic down pressure for 30 inches, or until refusal. Shelby Tubes were sealed with plastic caps and taped. Portions of the Shelby Tube samples were extruded for laboratory testing. Bulk samples were also collected from auger cuttings at selected intervals. Bulk samples are indicated on the boring logs with shading, Standard Penetration samples are indicated with vertical lines, and Shelby tube samples are indicated by diagonal lines.

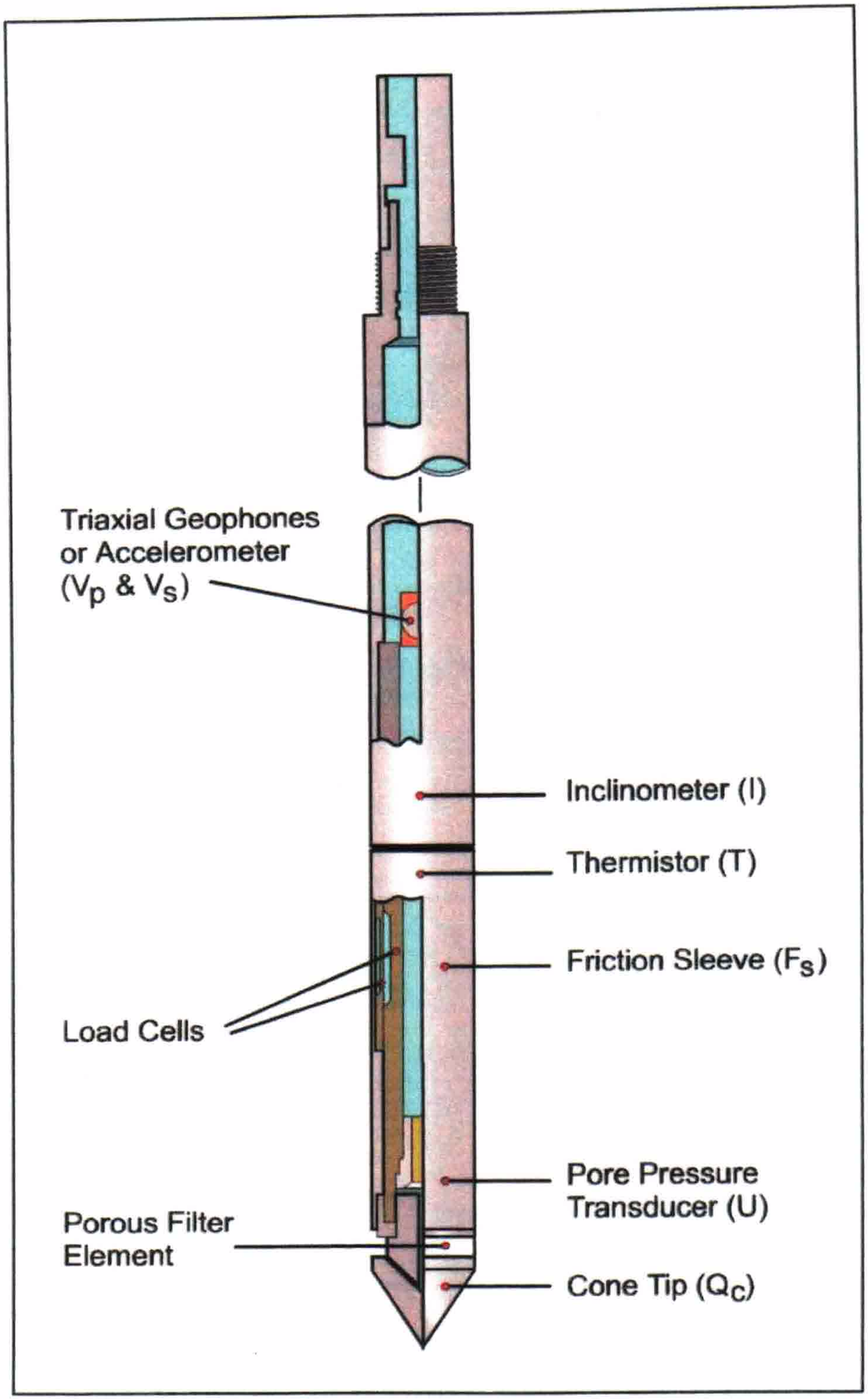
The 8-inch diameter hollow stem rig was also used to drill two percolation holes to an approximate depth of 3 feet each. The percolation holes were pre-saturated on November 29, 2001, and the percolation tests were conducted about 26 hours later in accordance with the County of San Diego standards for percolation testing.

The borings and CPT soundings were located by visually estimating and taping distances from landmarks shown on the Site Plan. The locations shown should not be considered more accurate than is implied by the method of measurement used. The lines designating the interface between differing soil materials on the boring logs indicate changes which may be abrupt or gradational.

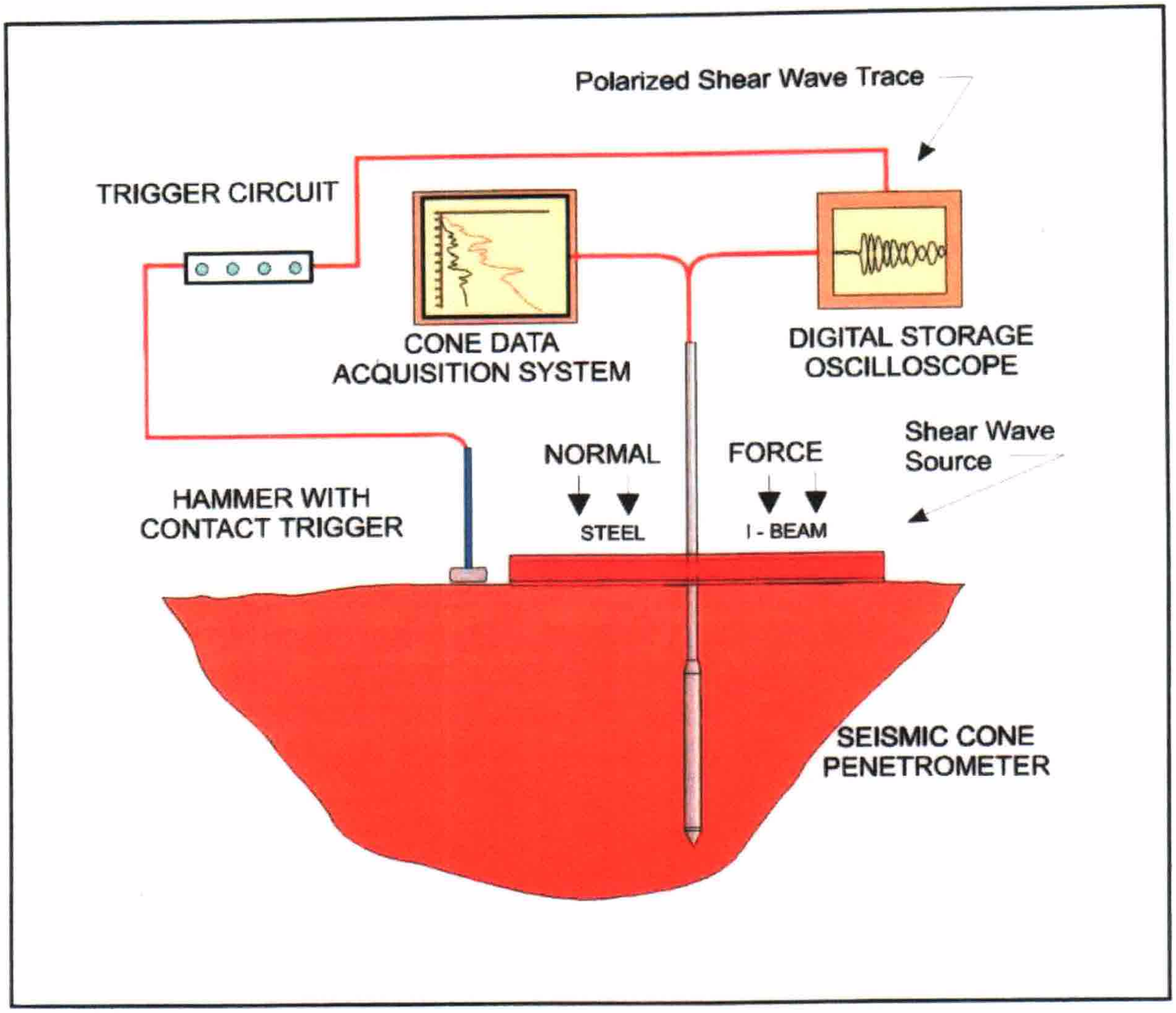
APPENDIX B

SUBSURFACE EXPLORATION (Continued)

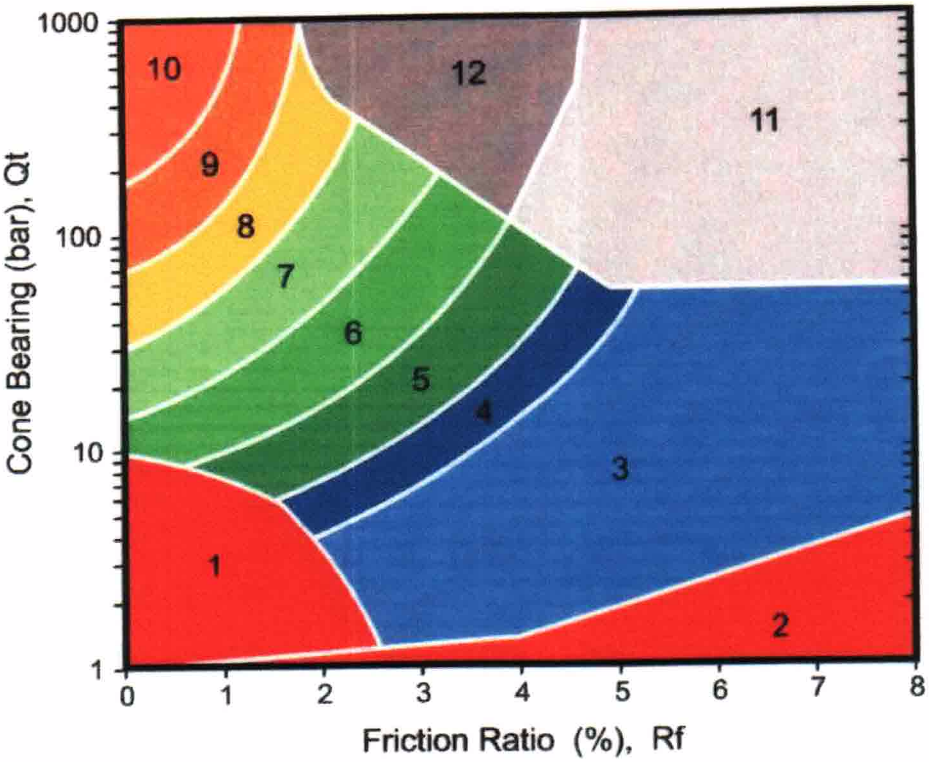
Further, soil conditions at locations between the excavations and soundings may be substantially different from those at the specific locations explored. Finally, it should be recognized that the passage of time can result in changes in the soil conditions reported in our logs.



CONE PENETROMETER DETAIL



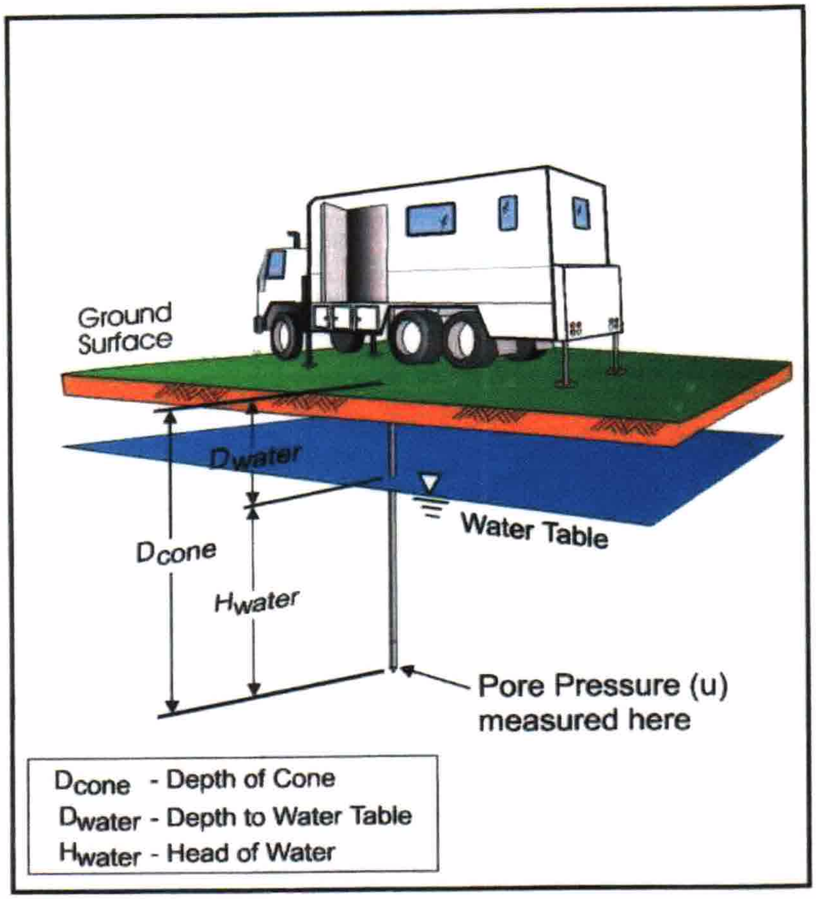
SHEAR WAVE EVALUATION

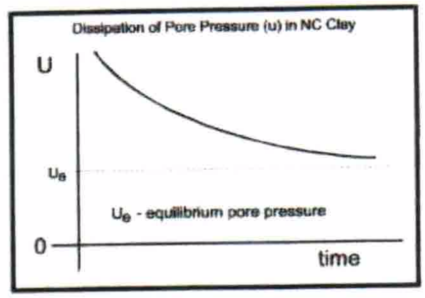


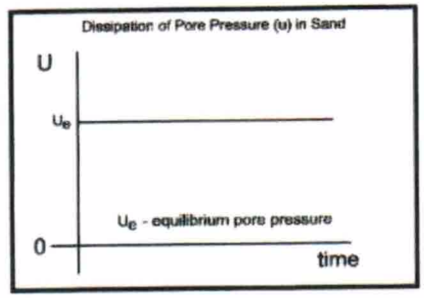
Zone	Q_t / N	Soil Behaviour Type
1	2	sensitive fine grained
2	1	organic material
3	1	clay
4	1.5	silty clay to clay
5	2	clayey silt to silty clay
6	2.5	sandy silt to clayey silt
7	3	silty sand to sandy silt
8	4	sand to silty sand
9	5	sand
10	6	gravelly sand to sand
11	1	very stiff fine grained *
12	2	sand to clayey sand *

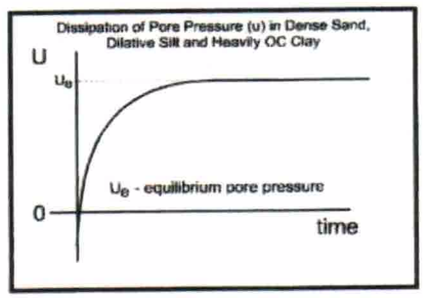
* overconsolidated or cemented

SOIL CLASSIFICATION









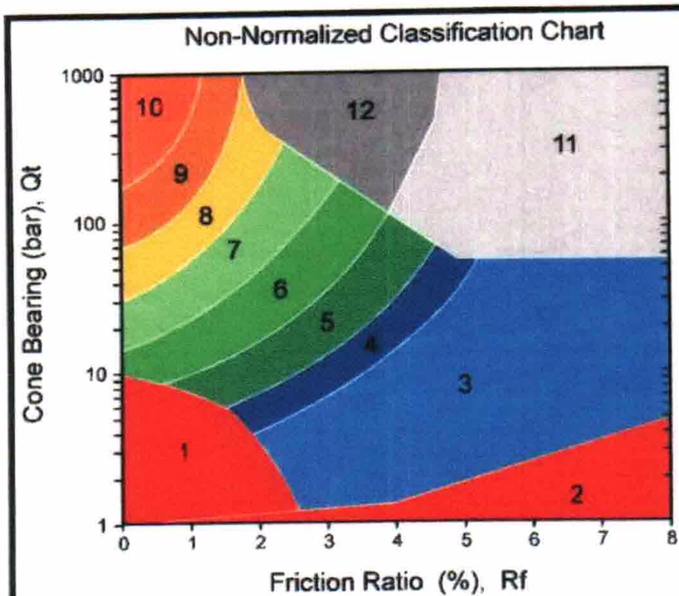
Water Table Calculation

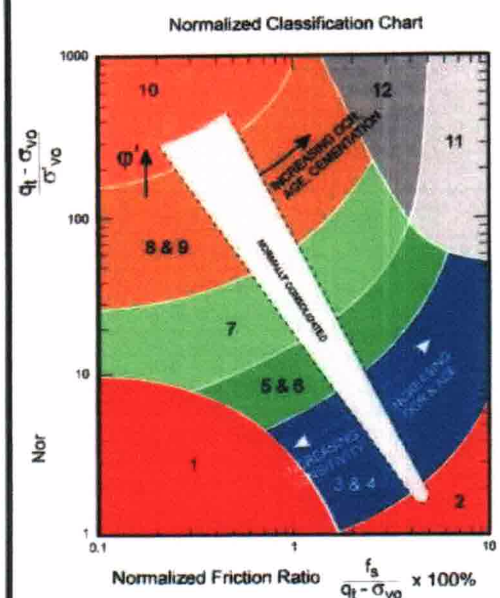
$$D_{water} = D_{cone} - H_{water}$$

where $H_{water} = U_e$ (depth units)

Useful Conversion Factors:
1psi = 0.704m = 2.31 feet (water)
1tsf = 0.958 bar = 13.9 psi
1m = 3.28 feet

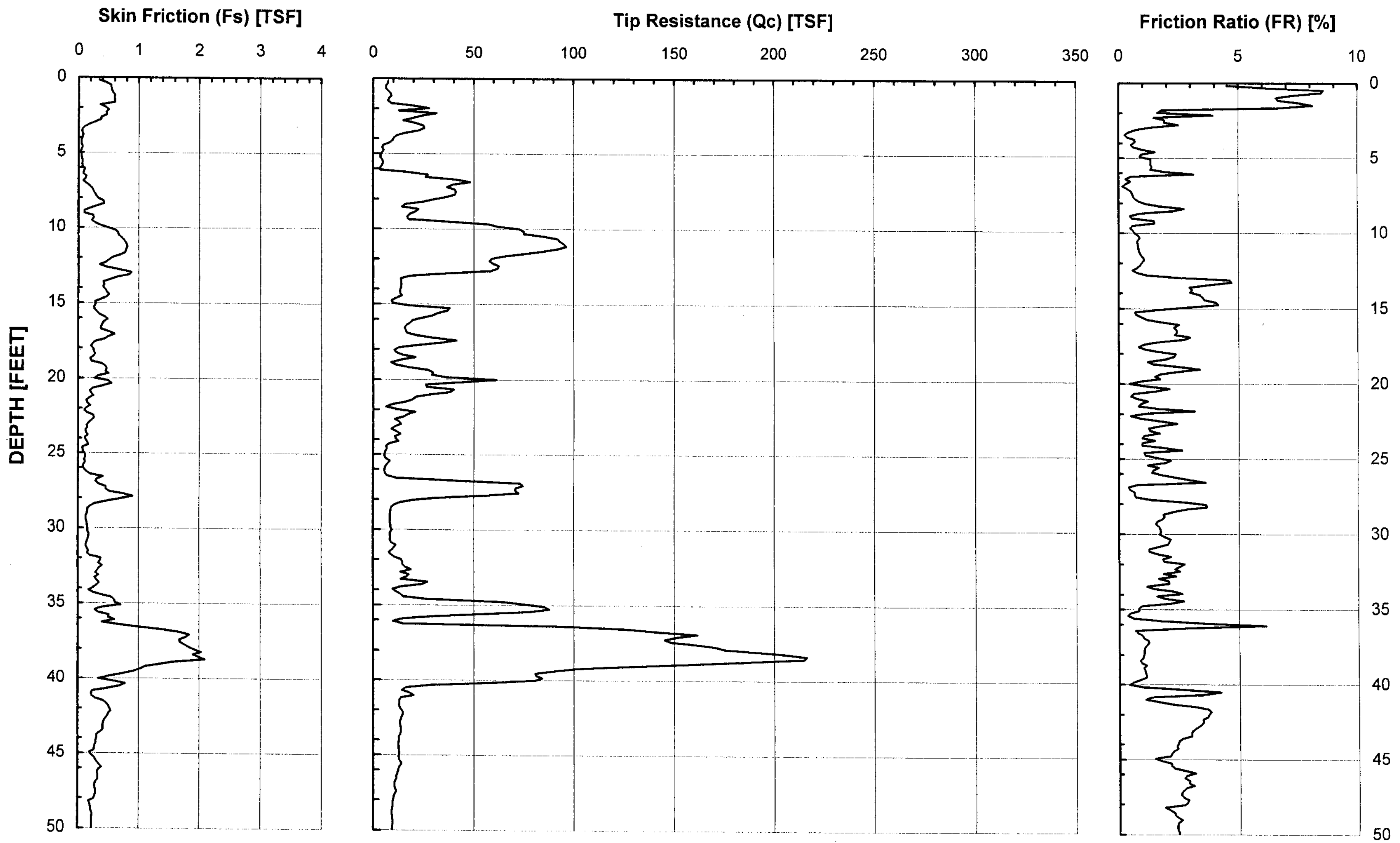
GROUND WATER EVALUATION





Zone		Q_t / N	Soil Behaviour Type
1		2	sensitive fine grained
2		1	organic material
3		1	clay
4		1.5	silty clay to clay
5		2	clayey silt to silty clay
6		2.5	sandy silt to clayey silt
7		3	
8		4	and to silty sand
9		5	sand
10		6	gravelly sand to sand
11		1	very stiff fine grained *
12		2	sand to clayey sand *

NON-NORMALIZED AND NORMALIZED SOIL BEHAVIOR TYPE CLASSIFICATION CHARTS



Geotechnics Incorporated

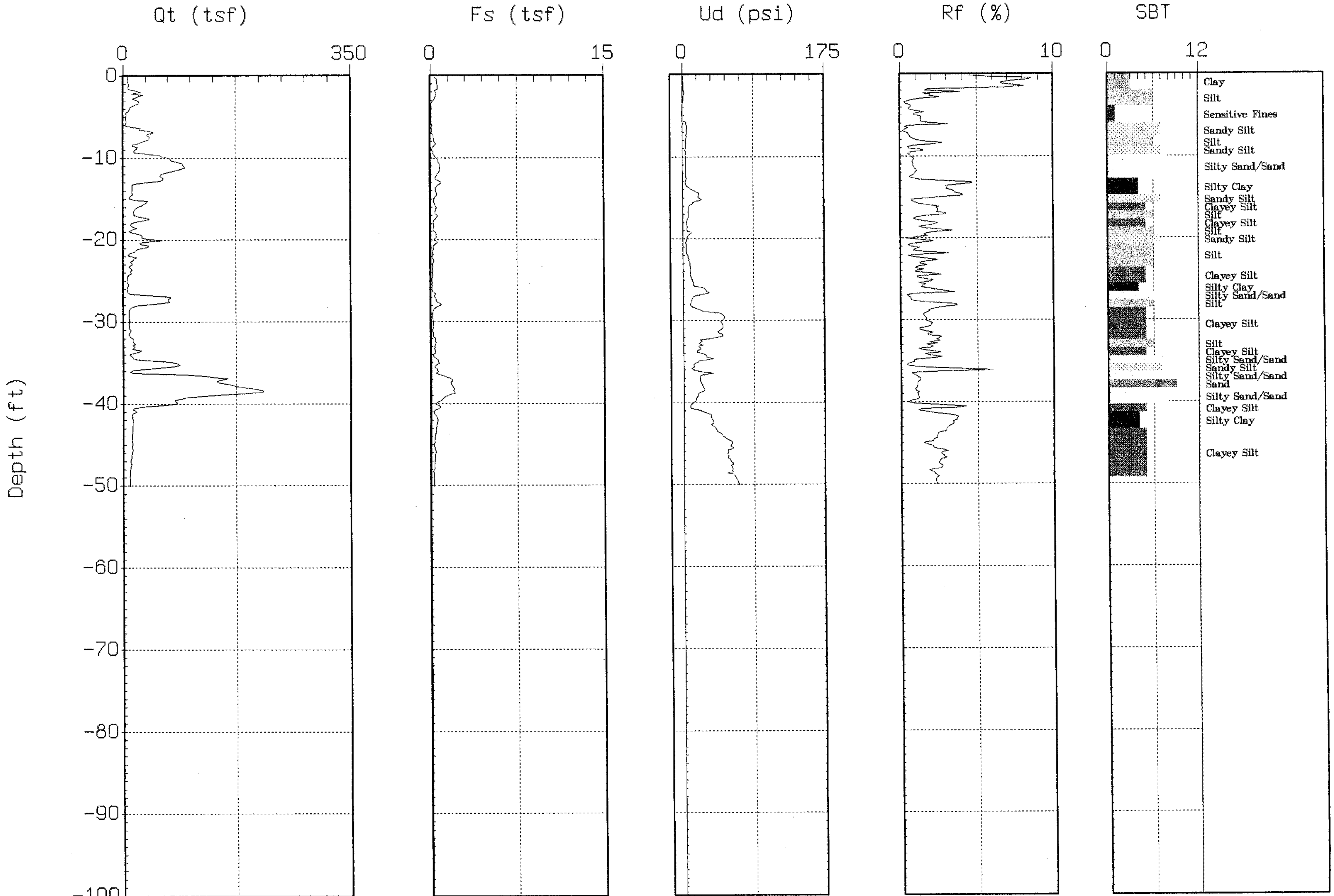
CONE PENETROMETER DATA (CPT-1)

Project No. 0673-002-00
Document No. 02-0022
FIGURE B-1

GEOTECHNICS

Site : CAL ENERGY UNIT
Location : CPT-01

Engineer : M. EDWARDS
Date : 12:03:01 15:47



Max. Depth: 50.03 (ft)

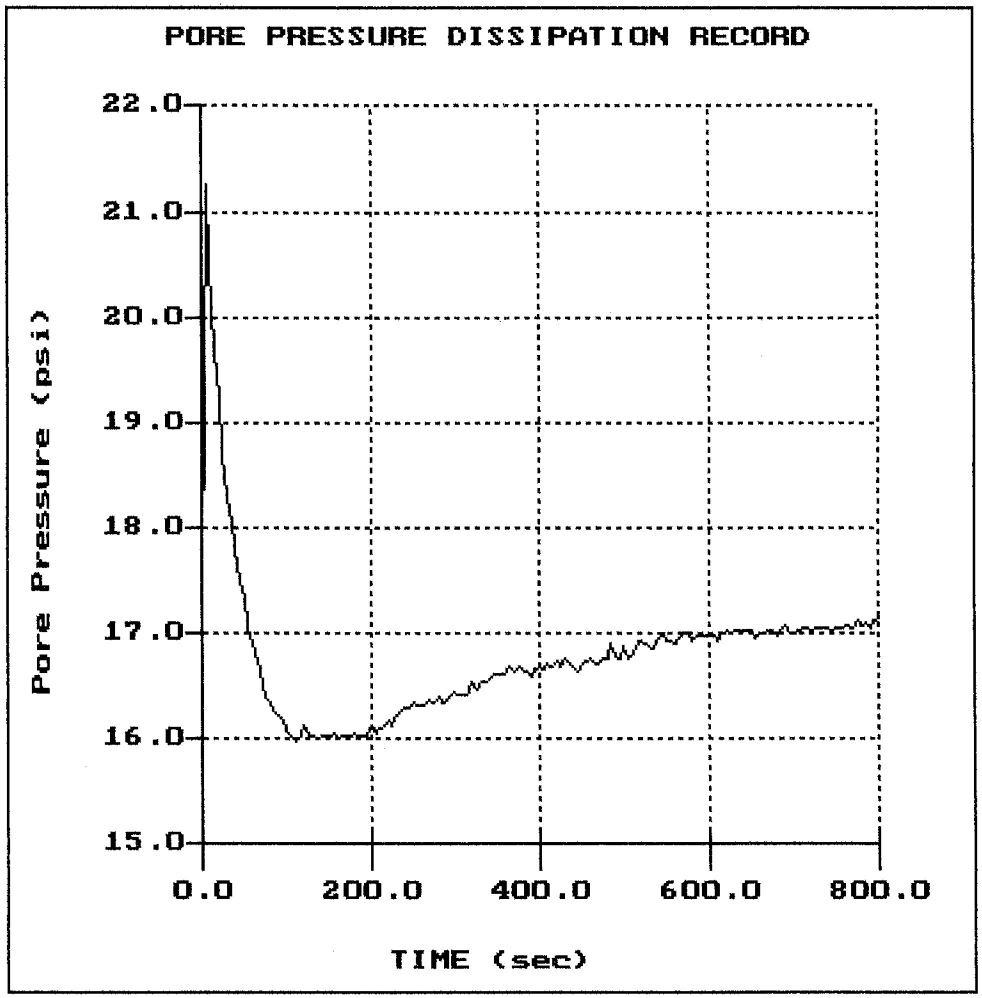
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson and Campanella 1988)

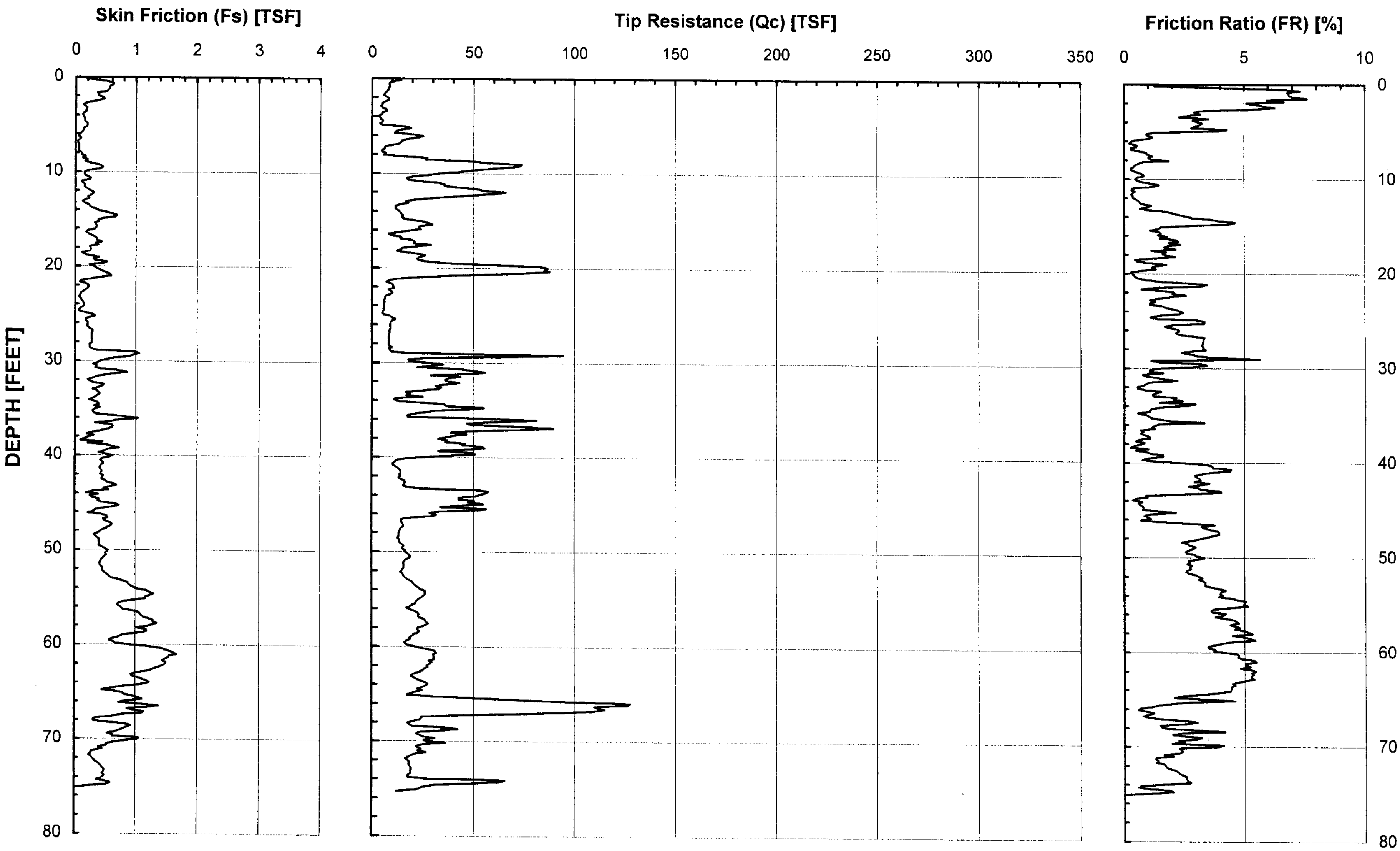
GEOTECHNICS

Site: CAL ENERGY UNIT 6
Location: CPT-01

Engineer: M. EDWARDS
Date:11:28:01 15:47



File: 326C01.PPC
Depth (m): 11.90
(ft): 39.04
Duration : 800.0s
U-min: 15.97 110.0s
U-max: 21.24 5.0s



Geotechnics Incorporated

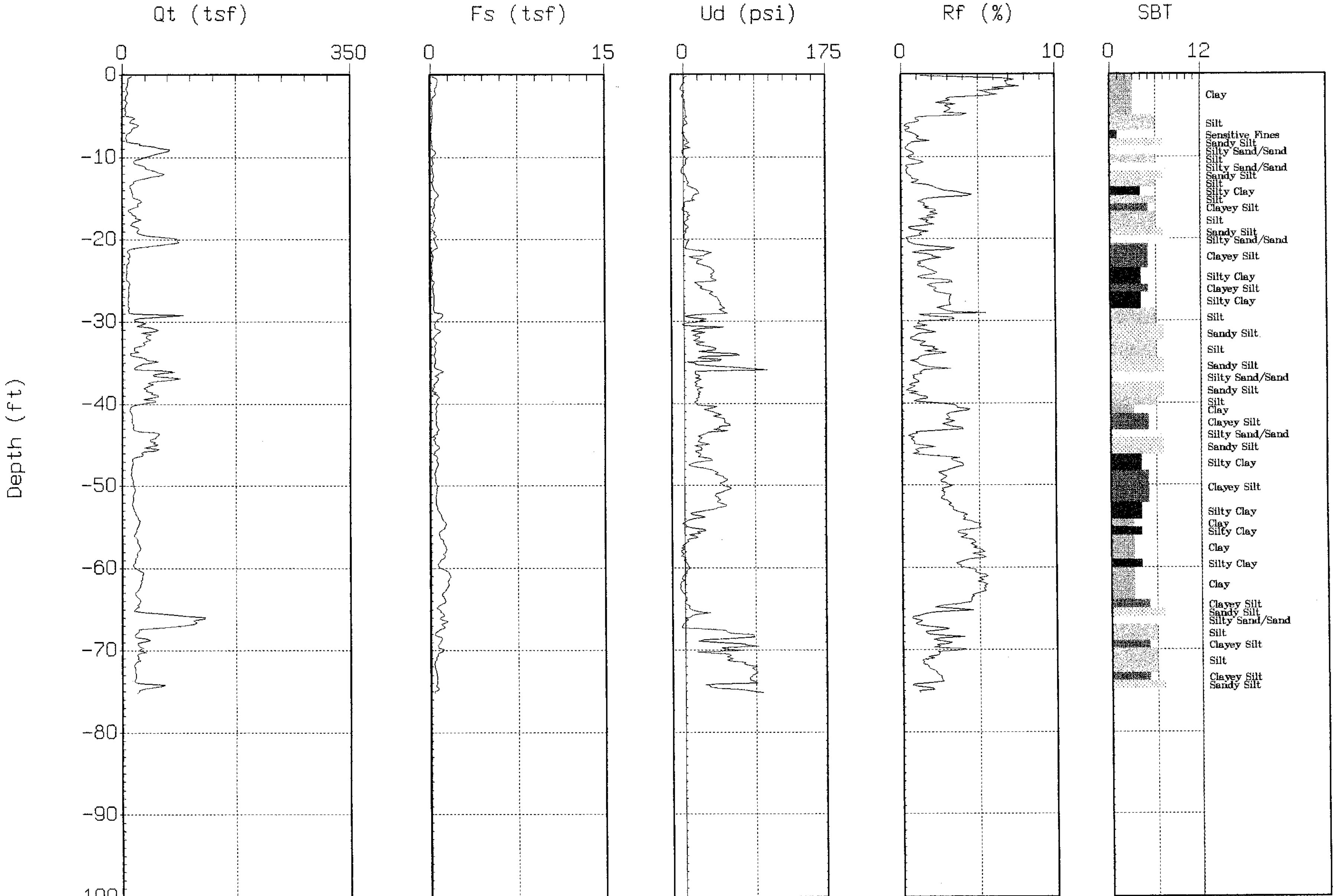
CONE PENETROMETER DATA (CPT-2)

Project No. 0673-002-00
Document No. 02-0022
FIGURE B-2

GREGG GEOTECHNICS

Site : CAL ENERGY UNIT
Location : SCPT-02

Engineer : M. EDWARDS
Date : 11:28:01 13:57



Max. Depth: 75.13 (ft)

Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson and Campanella 1988)

GREGG

GIS Project No: 01-326SH
Client: Geotechnics
Site: CAL ENERGY UNIT 6
Location SCPT-2
Sounding Date: December 3rd, 2001

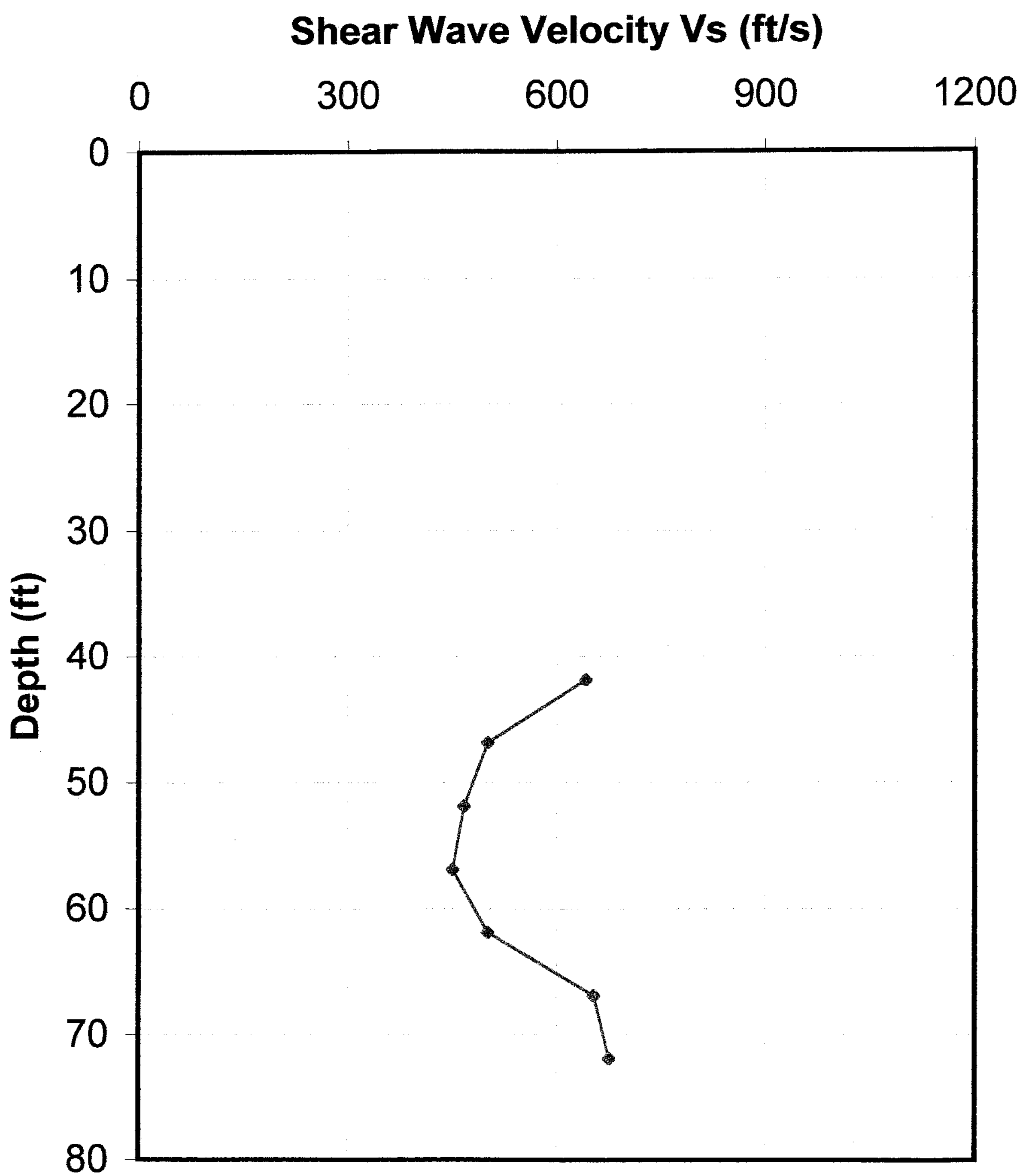
Shear Wave Velocity Calculations

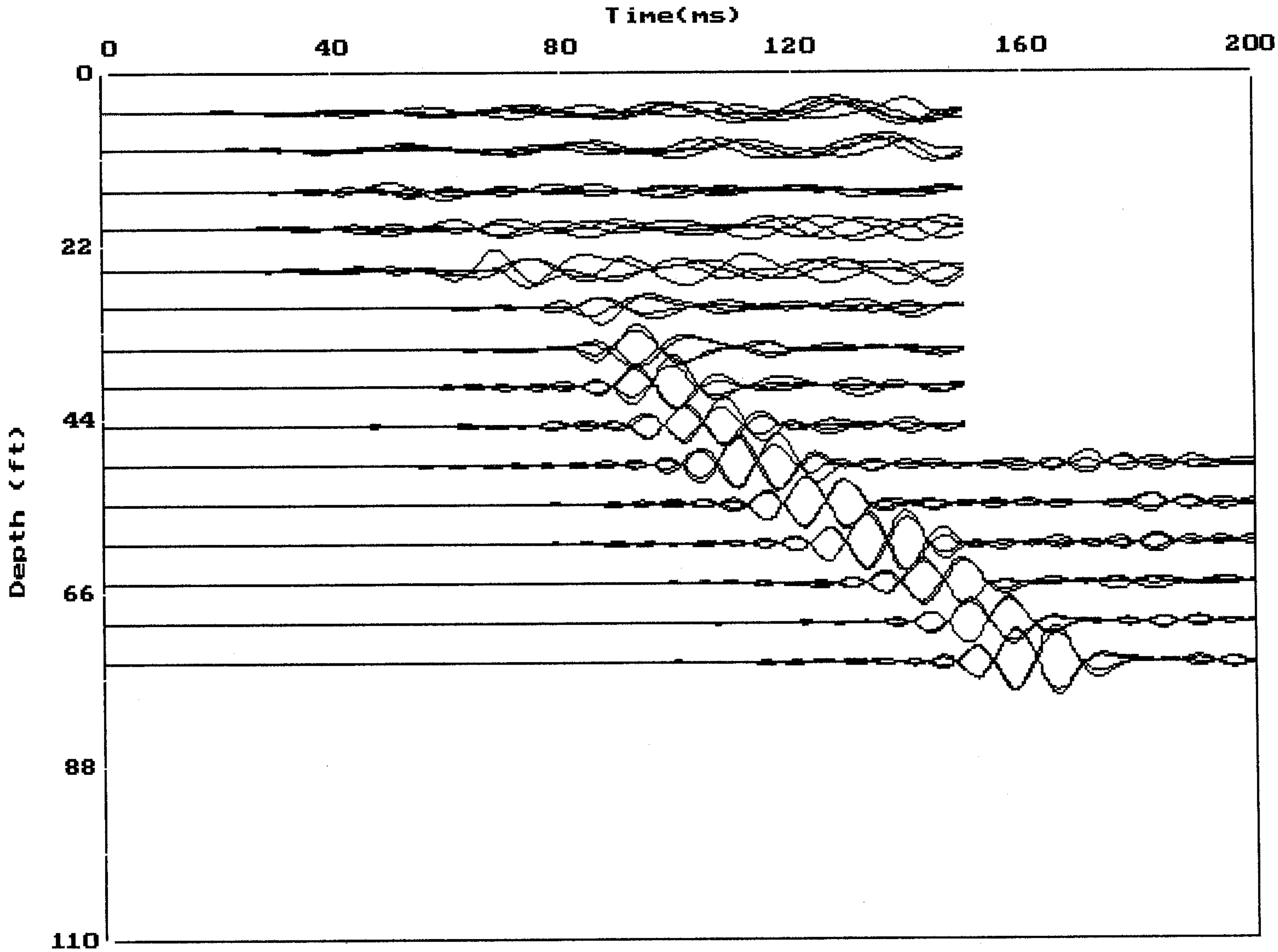
Geophone Offset	(feet)	0.66
Source Offset	(feet)	1.50

Test Depth (feet)	Geophone Depth (feet)	Ray Path (feet)	Incremental Distance (feet)	Time Interval (ms)	Interval Velocity (ft/s)	Interval Depth (feet)
40.02	39.36	39.39				
45.10	44.44	44.47	5.08	7.90	643	41.90
50.00	49.35	49.37	4.90	9.77	501	46.89
55.10	54.44	54.46	5.10	10.90	468	51.89
60.02	59.36	59.38	4.92	10.91	451	56.90
65.11	64.45	64.47	5.09	10.15	501	61.91
70.03	69.37	69.39	4.92	7.52	654	66.91
75.11	74.45	74.47	5.08	7.51	676	71.91

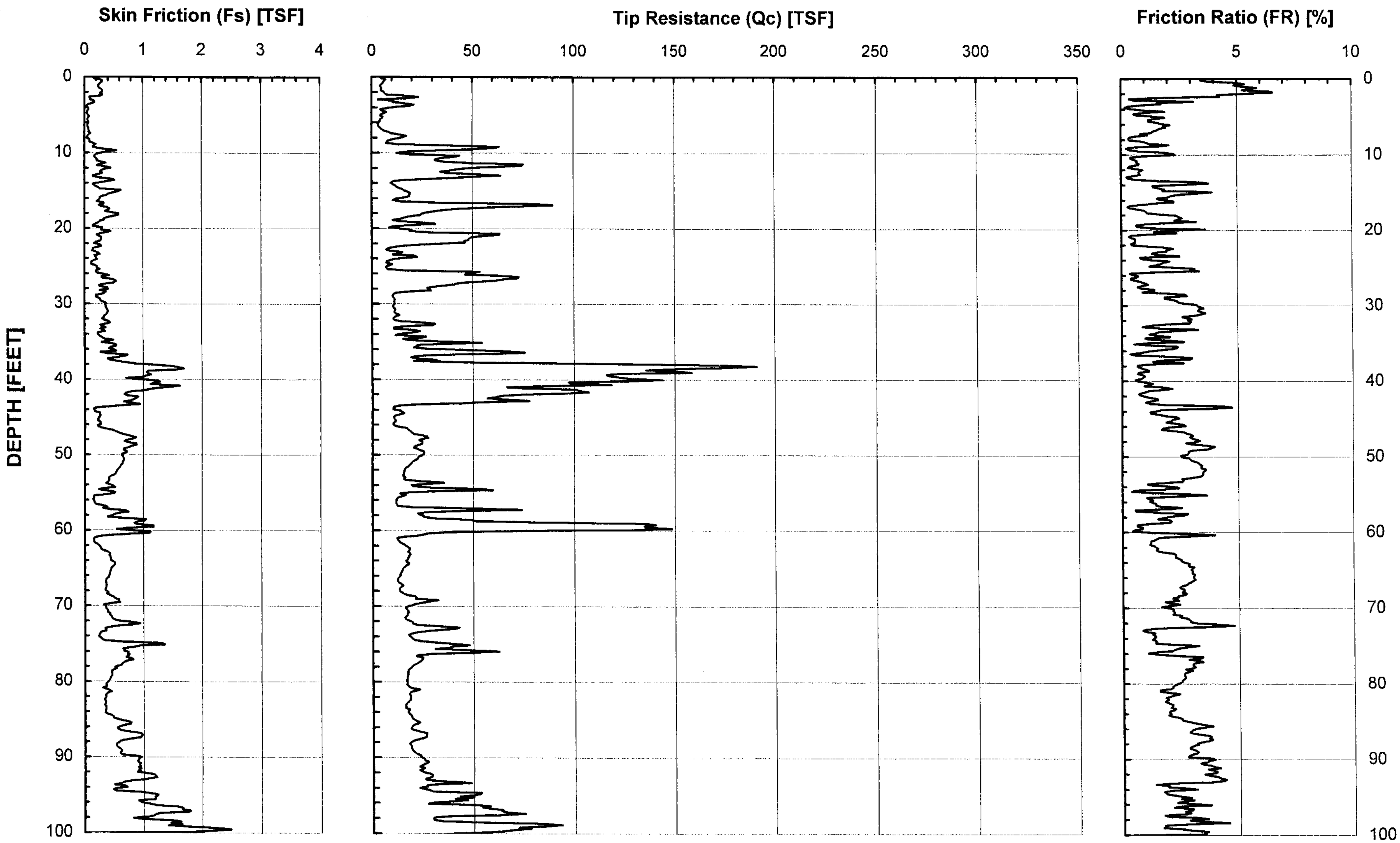
Shear Wave Velocity Profile

Client:Geotechnics
Site:Cal Energy Unit 6
Location: SCPT-2
Sounding Date: December 3rd, 2001





GEOTECHNICS CAL ENERGY UNIT 6 SCPT-2 12/03/01



Geotechnics Incorporated

CONE PENETROMETER DATA (CPT-3)

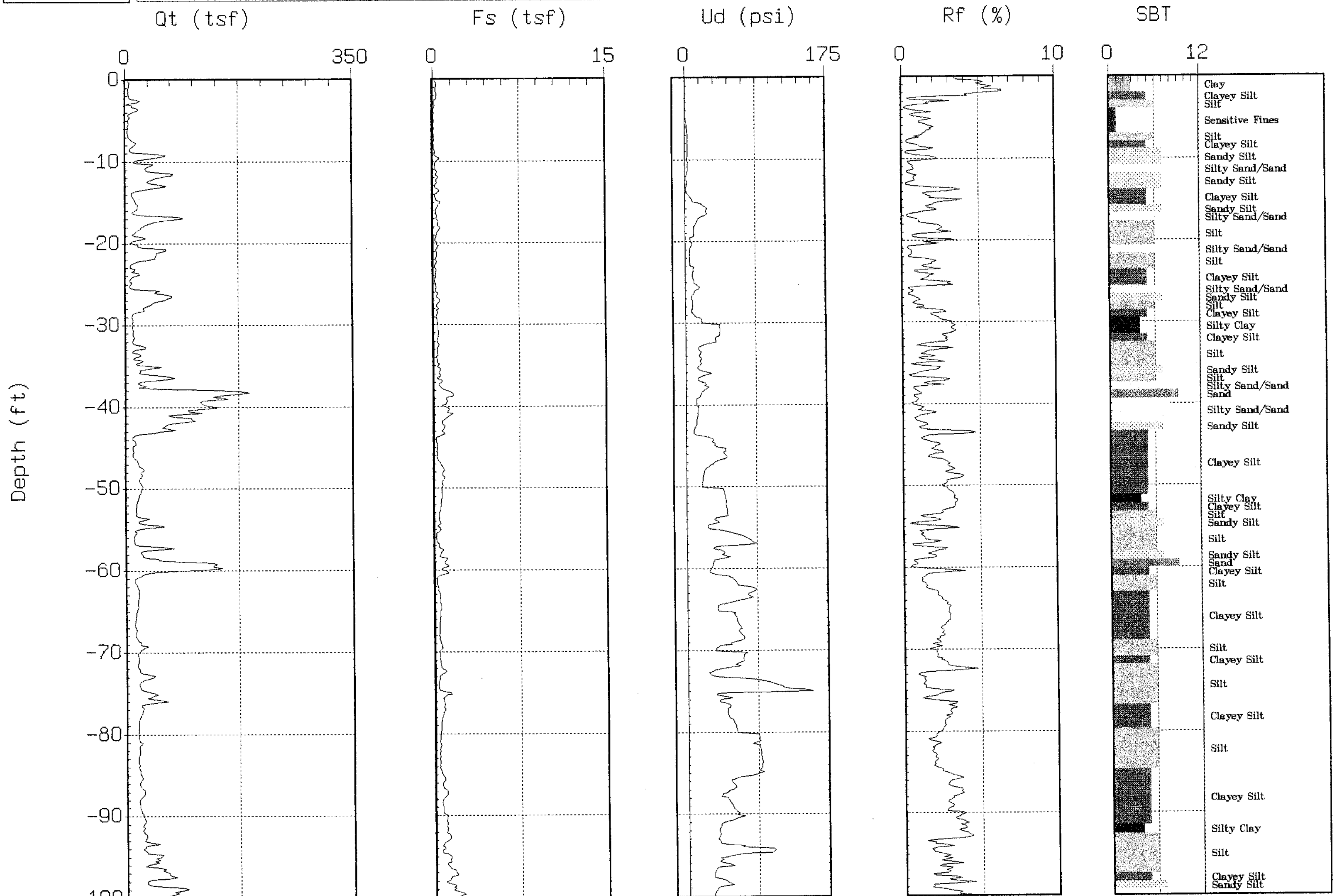
Project No. 0673-002-00
Document No. 02-0022
FIGURE B-3

GREGG

GEOTECHNICS

Site : CAL ENERGY UNIT
Location : SCPT-03

Engineer : M. EDWARDS
Date : 12:03:01 16:22



Max. Depth: 100.06 (ft)

Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson and Campanella 1988)

GREGG

GIS Project No: 01-326SH
Client: Geotechnics
Site: Cal Energy Unit 6
Location SCPT-3
Sounding Date: December 3, 2001

Shear Wave Velocity Calculations

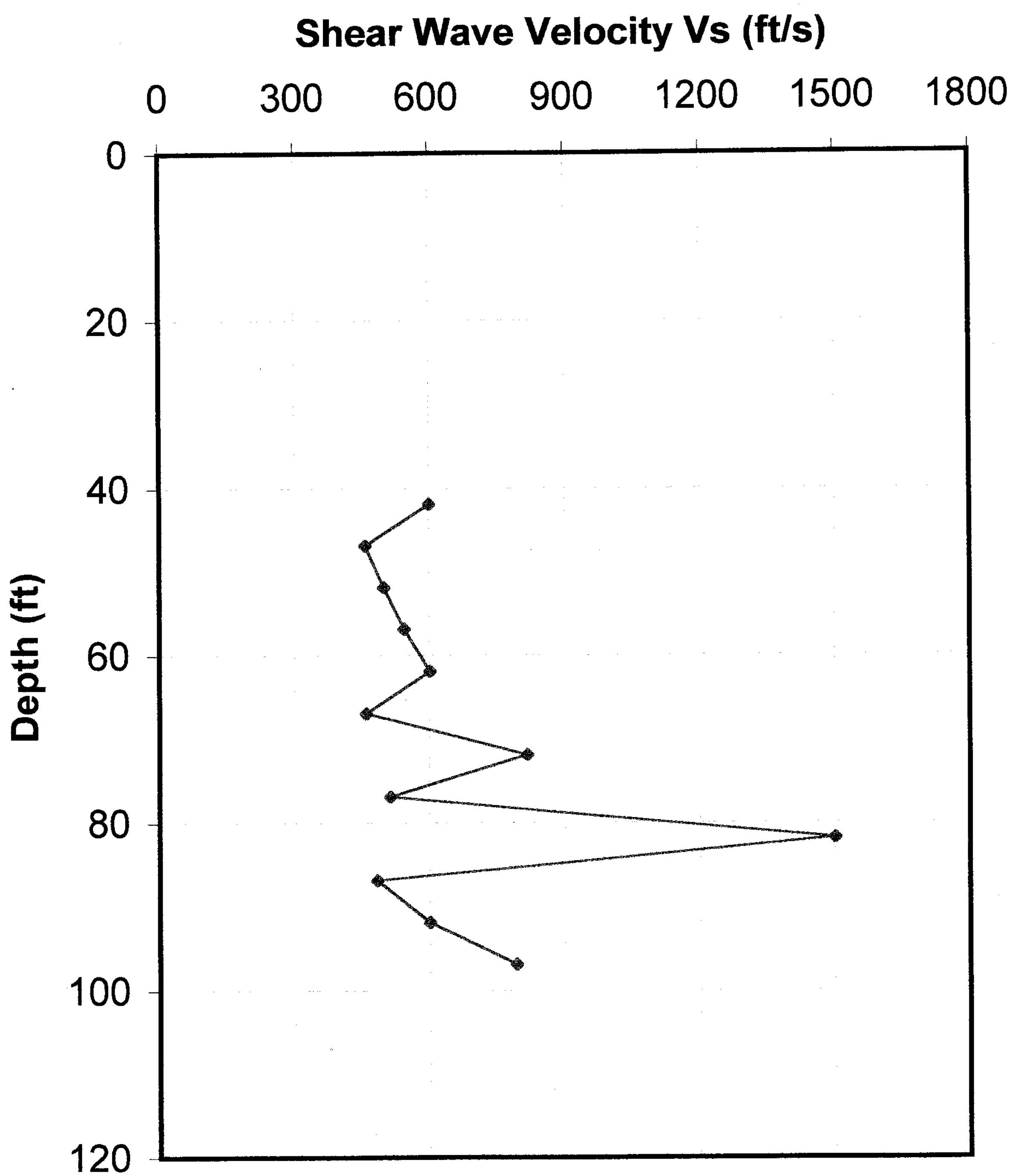
Geophone Offset (feet) 0.66
Source Offset (feet) 1.50

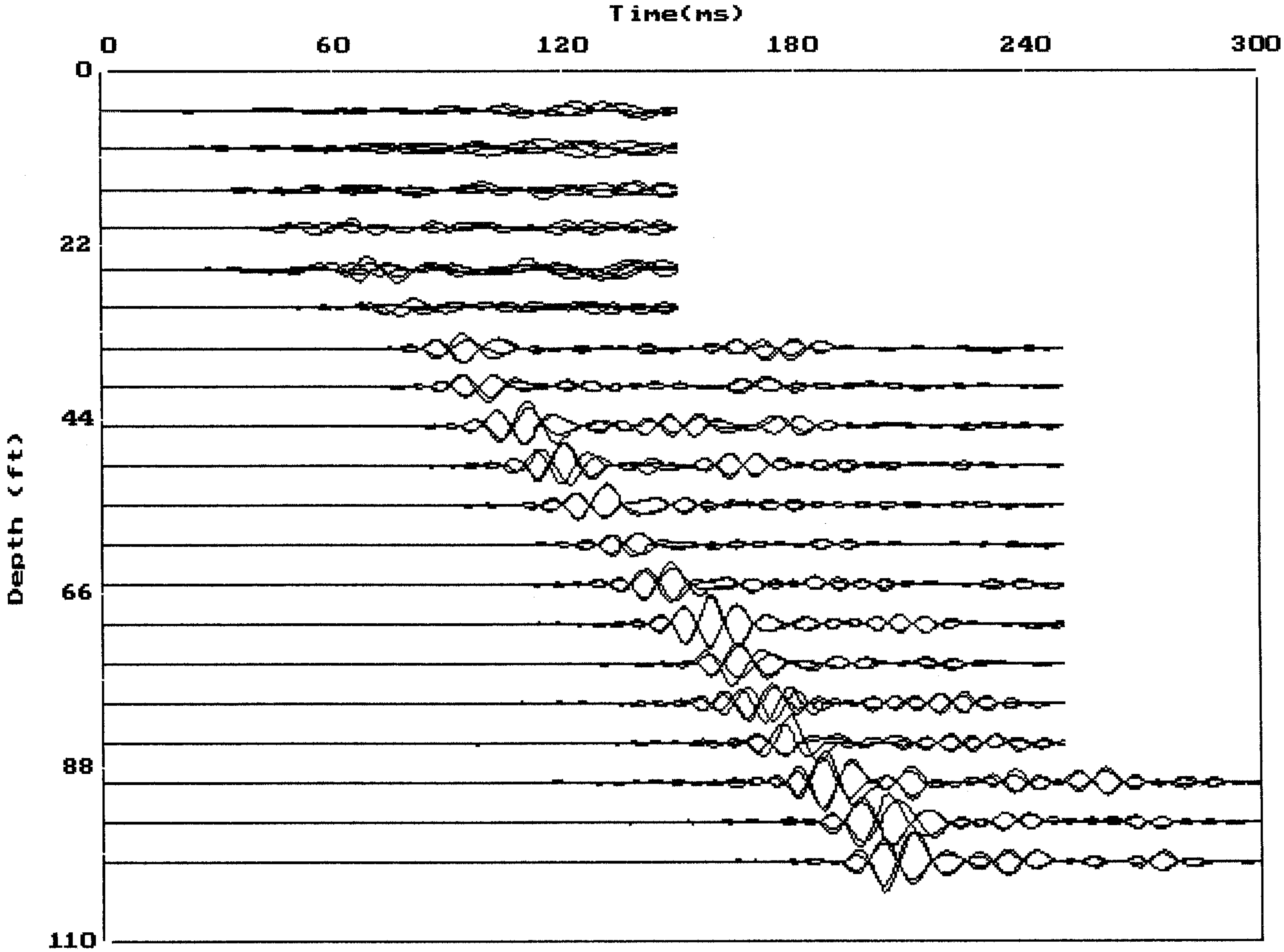
Test Depth (feet)	Geophone Depth (feet)	Ray Path (feet)	Incremental Distance (feet)	Time Interval (ms)	Interval Velocity (ft/s)	Interval Depth (feet)
40.02	39.36	39.39				
45.10	44.44	44.47	5.08	8.45	601	41.90
50.02	49.36	49.39	4.92	10.72	459	46.90
55.10	54.44	54.46	5.08	10.15	500	51.90
60.02	59.36	59.38	4.92	9.02	545	56.90
65.11	64.45	64.47	5.09	8.46	601	61.91
70.03	69.37	69.39	4.92	10.71	459	66.91
75.11	74.45	74.47	5.08	6.21	818	71.91
80.03	79.37	79.39	4.92	9.58	513	76.91
85.12	84.46	84.48	5.09	3.39	1501	81.92
90.04	89.38	89.40	4.92	10.15	485	86.92
95.12	94.46	94.48	5.08	8.46	600	91.92
100.04	99.38	99.40	4.92	6.20	793	96.92

Shear Wave Velocity Profile

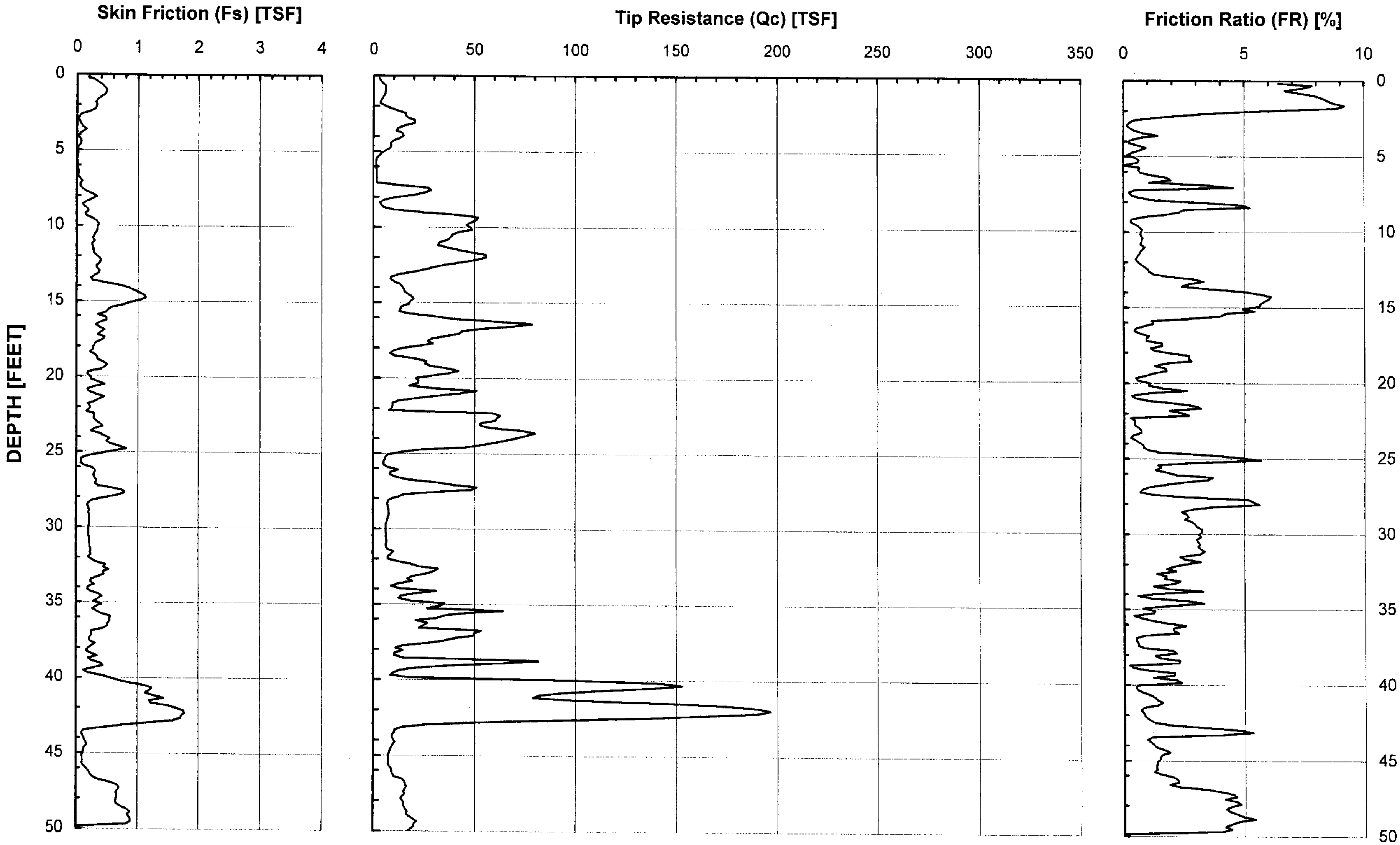
GREGG

Client: Geotechnics
Site:Cal Energy Unit 6
Location: SCPT-3
Sounding Date: December 3rd, 2001





GEOTECHNICS CAL ENERGY PLANT UNIT 6 SCPT-3



Geotechnics Incorporated

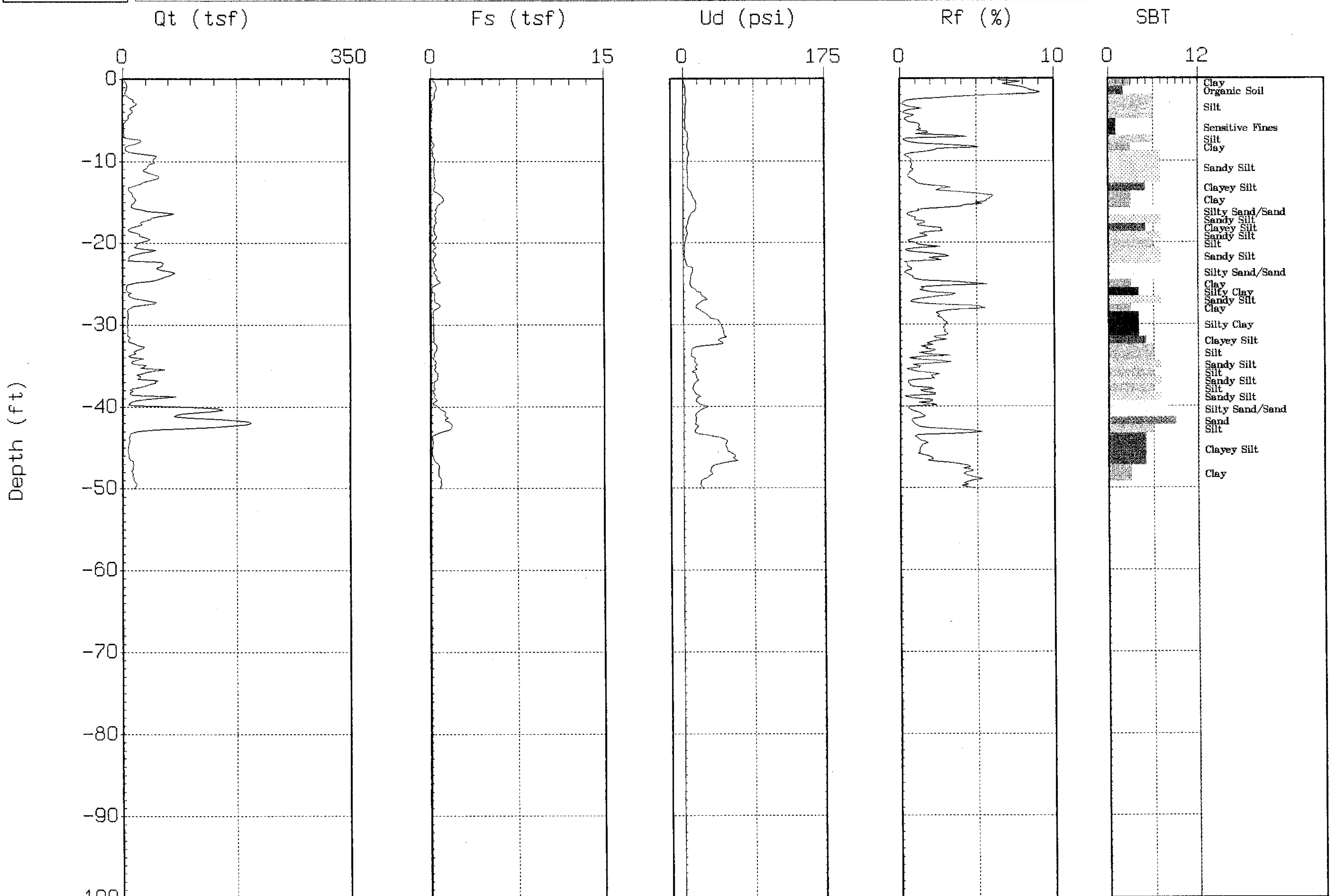
CONE PENETROMETER DATA (CPT-4)

Project No. 0673-002-00
Document No. 02-0022
FIGURE B-4

GREGG GEOTECHNICS

Site : CAL ENERGY UNIT
Location : CPT-04

Engineer : M.EDWARDS
Date : 12:03:01 18:01



Max. Depth: 50.03 (ft)

Depth Inc.: 0.164 (ft)

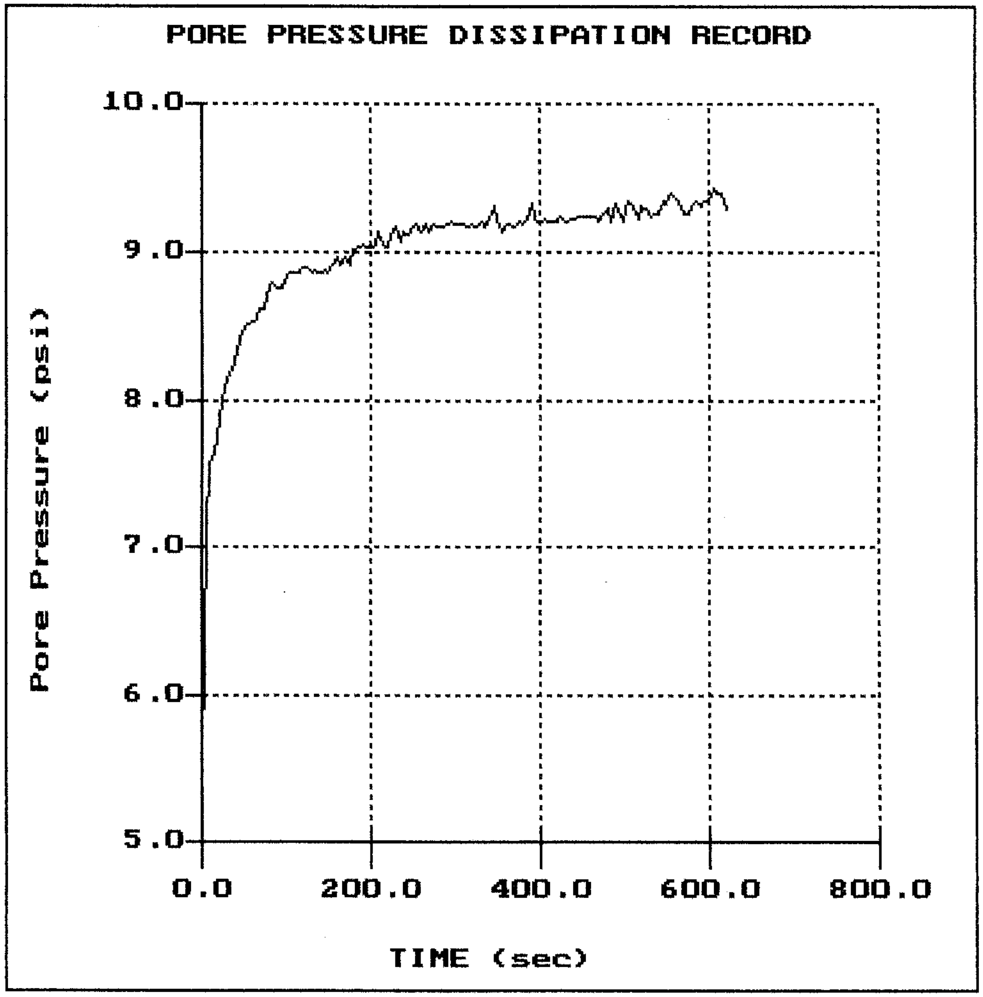
SBT: Soil Behavior Type (Robertson and Campanella 1988)

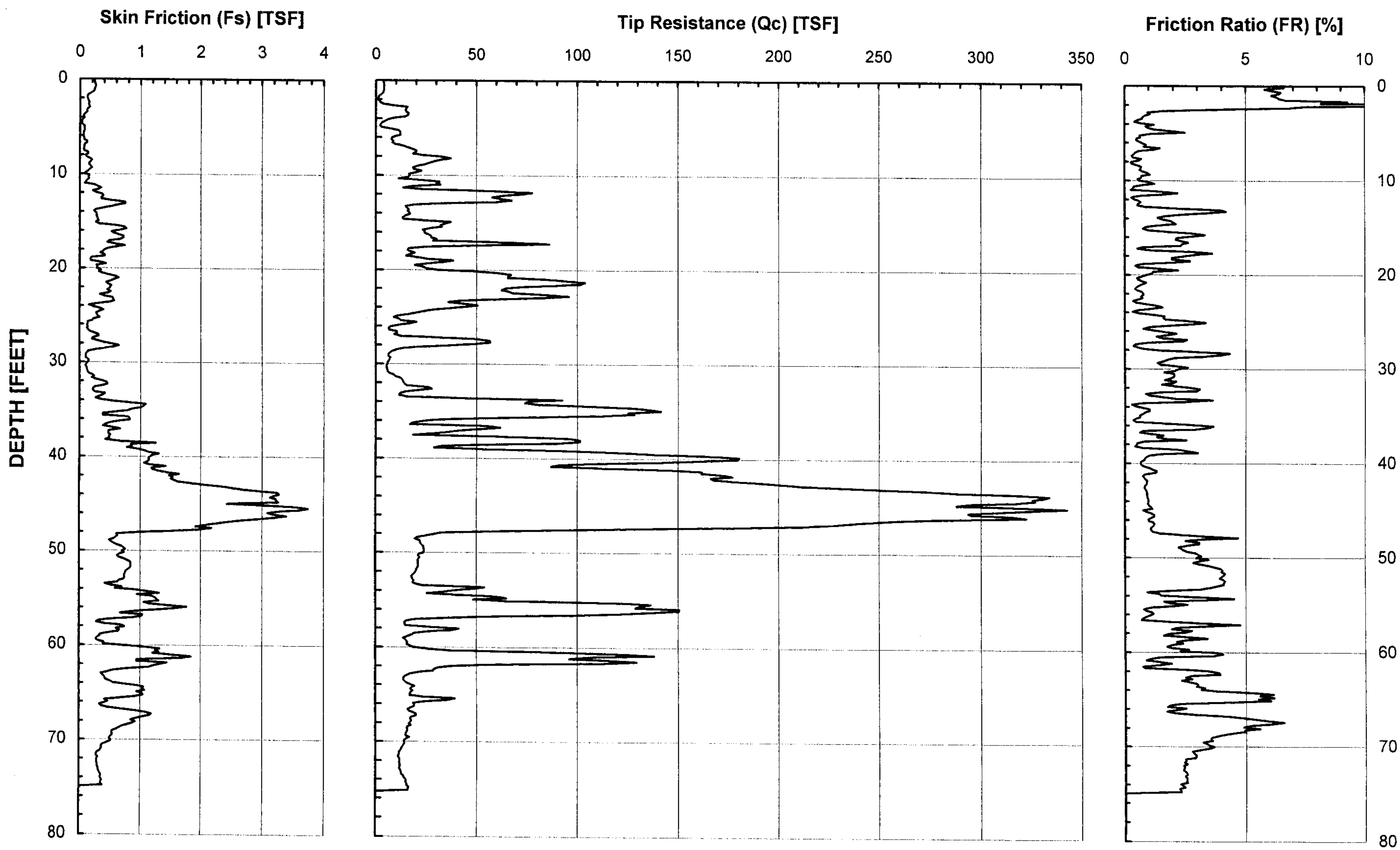
GEOTECHNICS

Site: CAL ENERGY UNIT 6
Location: CPT-04

Engineer: M. EDWARDS
Date:11:28:01 18:01

File: 326C04.PPC
Depth (m): 6.95
(ft): 22.80
Duration : 620.0s
U-min: 5.49 0.0s
U-max: 9.42 605.0s





Geotechnics Incorporated

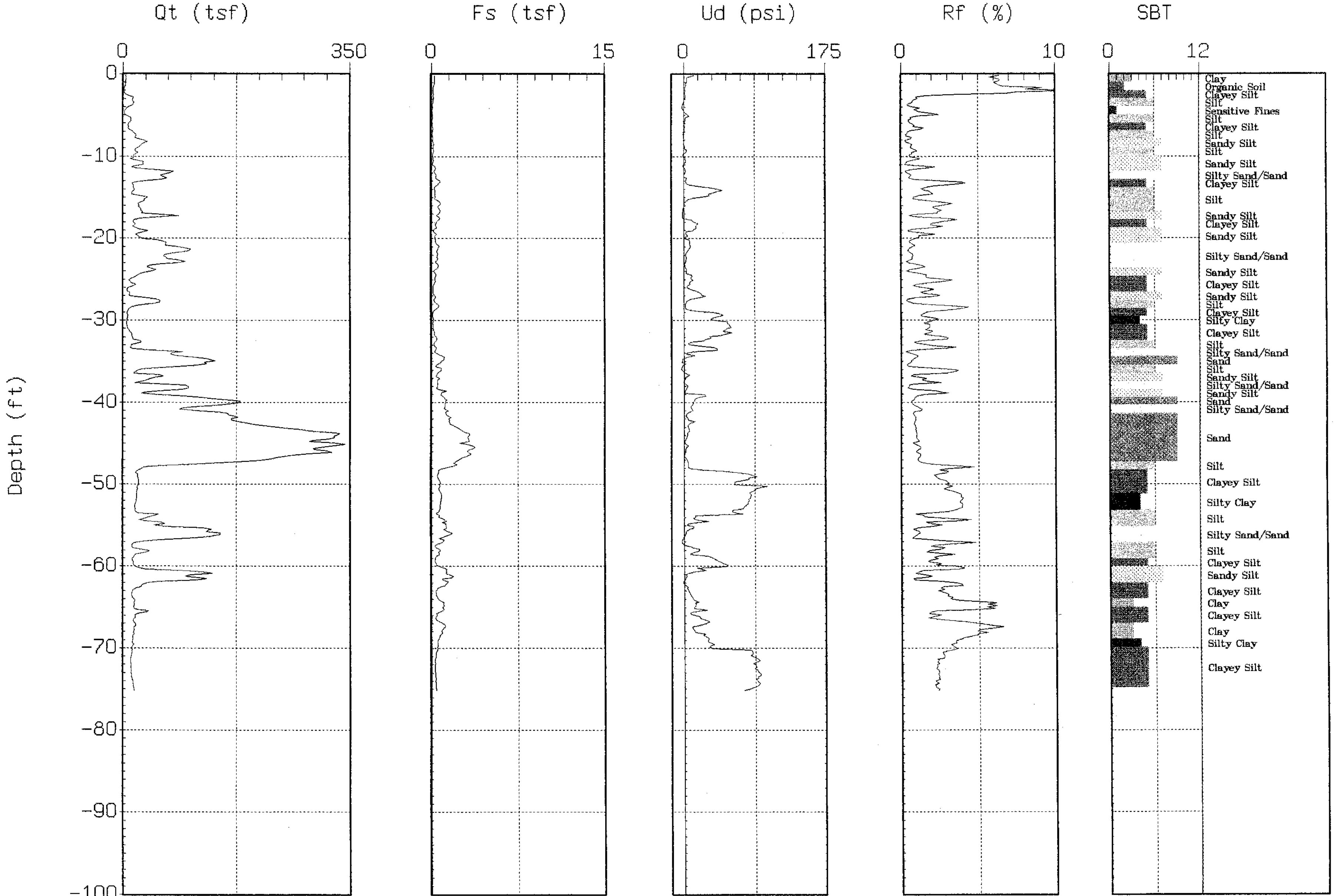
CONE PENETROMETER DATA (CPT-5)

Project No. 0673-002-00
Document No. 02-0022
FIGURE B-5

GREGG GEOTECHNICS

Site : CAL ENERGY UNIT
Location : SCPT-05

Engineer : M. EDWARDS
Date : 12:04:01 21:49



Max. Depth: 75.13 (ft)

Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson and Campanella 1988)

GIS Project No: 01-326SH
Client: Geotechnics
Site: Cal Energy Unit 6
Location SCPT-05
Sounding Date: Dec. 3, 2001

Shear Wave Velocity Calculations

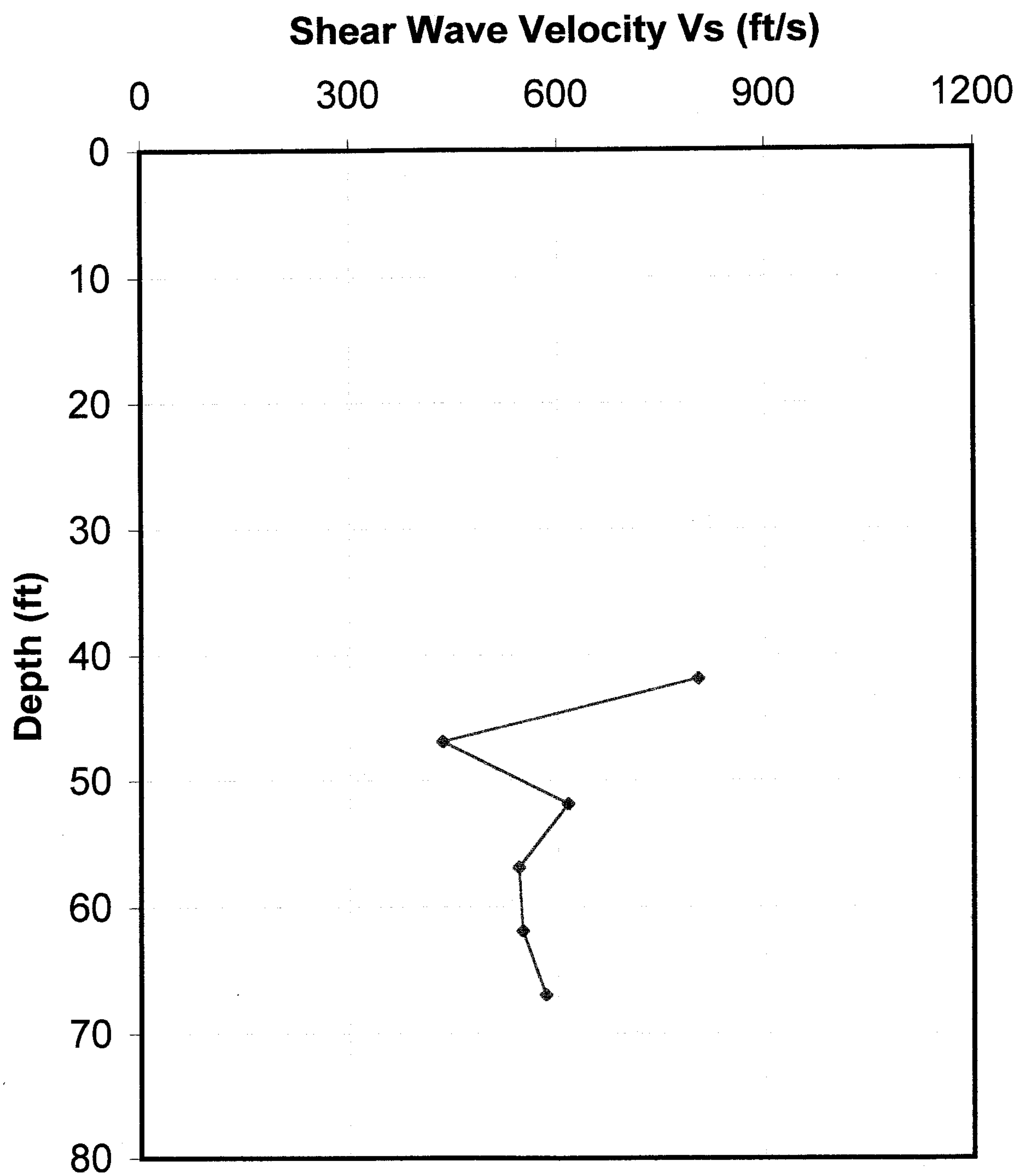
Geophone Offset (feet) 0.66
Source Offset (feet) 1.50

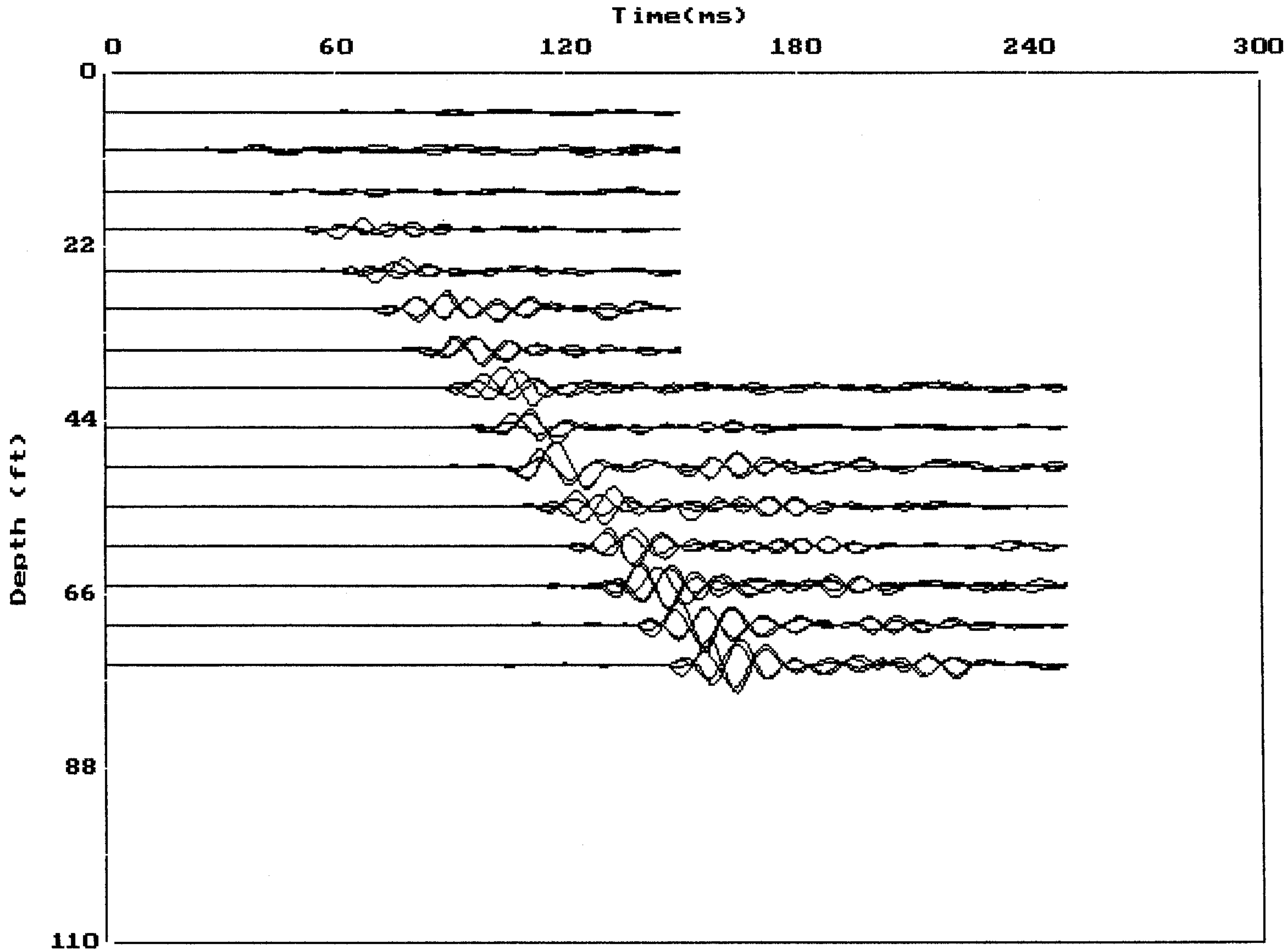
Test Depth (feet)	Geophone Depth (feet)	Ray Path (feet)	Incremental Distance (feet)	Time Interval (ms)	Interval Velocity (ft/s)	Interval Depth (feet)
40.02	39.36	39.39				
45.10	44.44	44.47	5.08	6.32	803	41.90
50.02	49.36	49.39	4.92	11.31	435	46.90
55.10	54.44	54.46	5.08	8.25	615	51.90
60.02	59.36	59.38	4.92	9.04	544	56.90
65.11	64.45	64.47	5.09	9.25	550	61.91
70.03	69.37	69.39	4.92	8.44	583	66.91

Shear Wave Velocity Profile

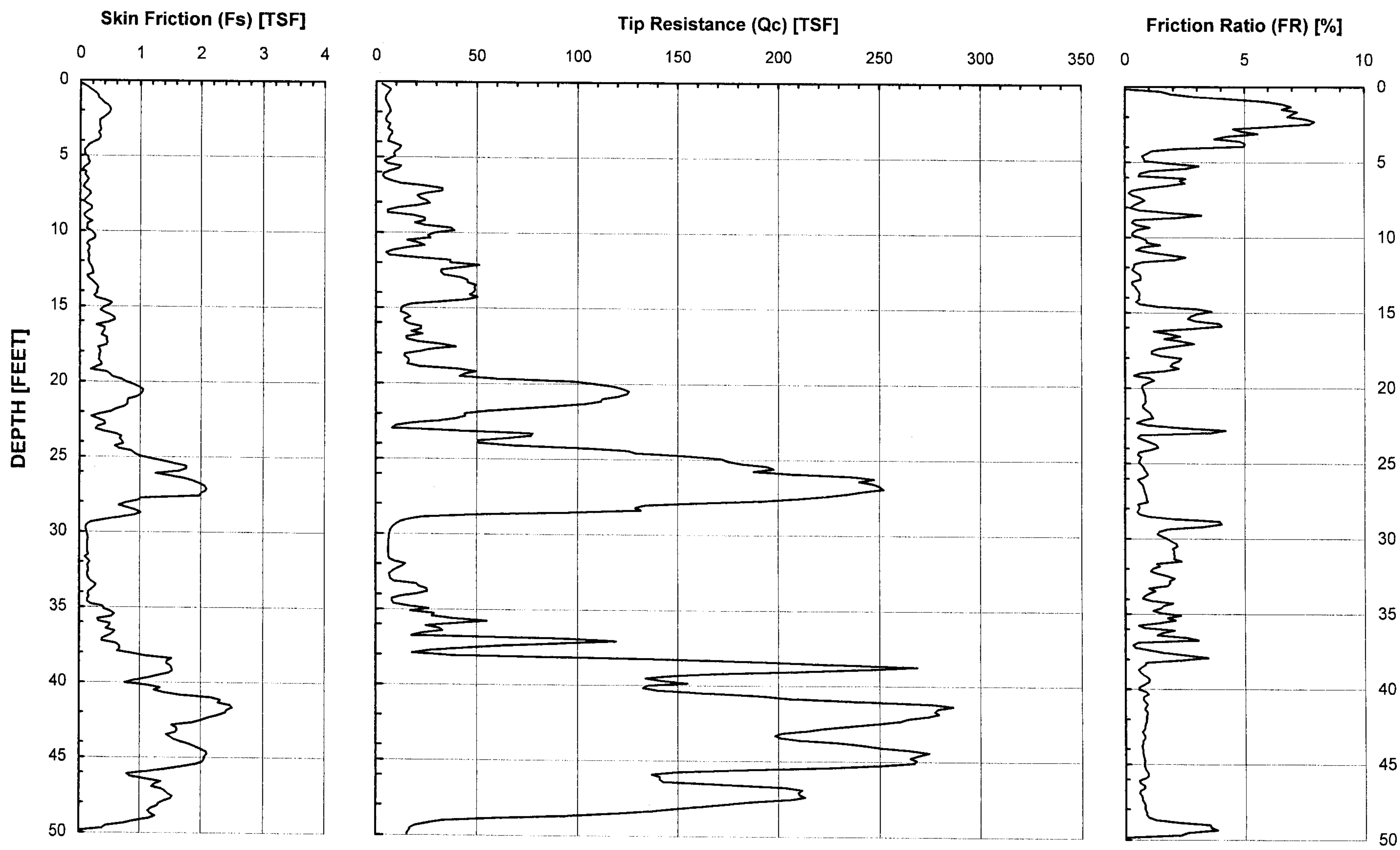
GREGG

Client: Geotechnics
Site:Cal Energy Unit 6
Location: SCPT-5
Sounding Date: December 3rd, 2001





GEOTECHNICS CAL ENERGY PLANT UNIT 6 SCPT-5



Geotechnics Incorporated

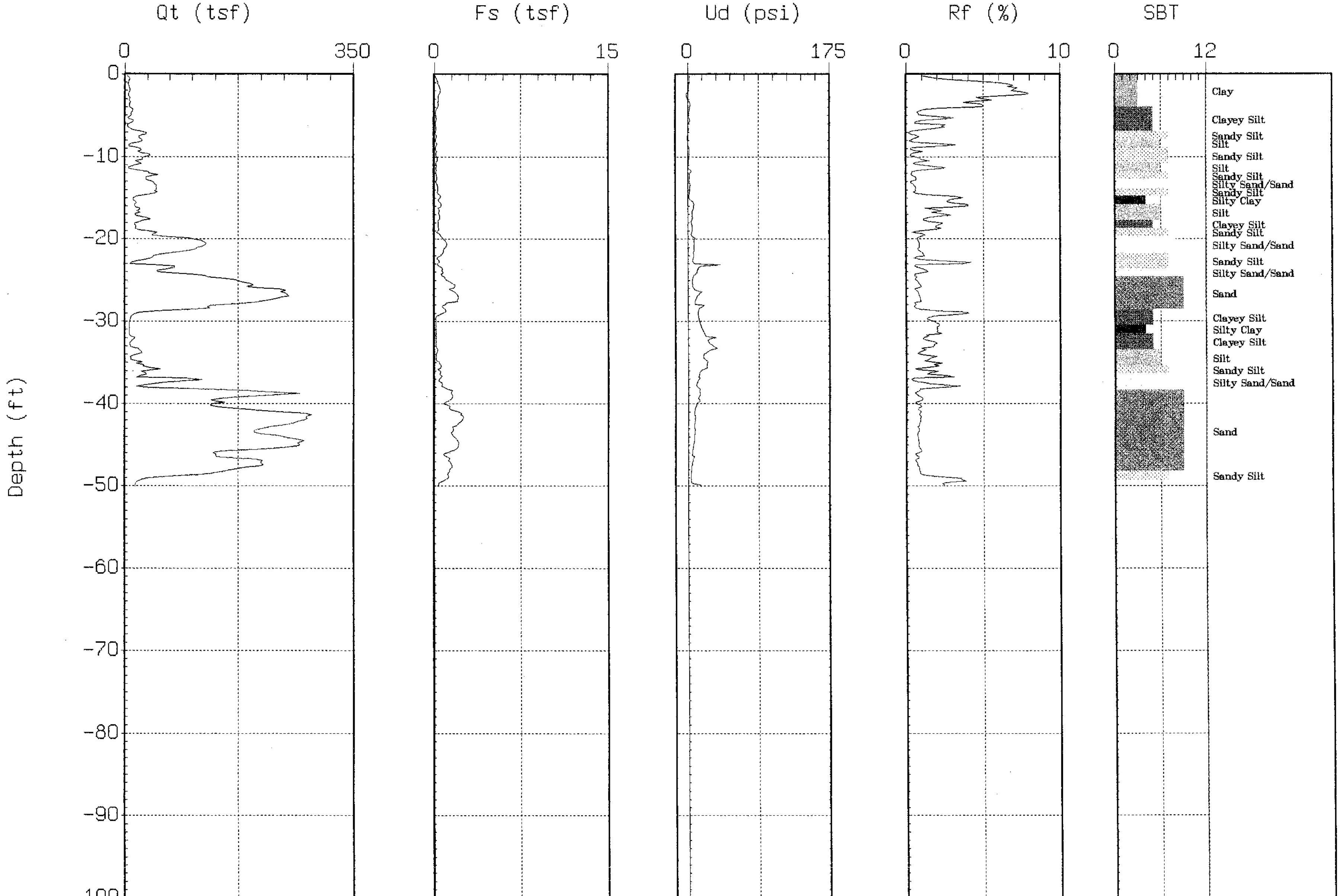
CONE PENETROMETER DATA (CPT-6)

Project No. 0673-002-00
Document No. 02-0022
FIGURE B-6

GEOTECHNICS

Site : CAL ENERGY UNIT
Location : CPT-06

Engineer : M. EDWARDS
Date : 12:03:01 18:55



Max. Depth: 50.03 (ft)

Depth Inc.: 0.164 (ft)

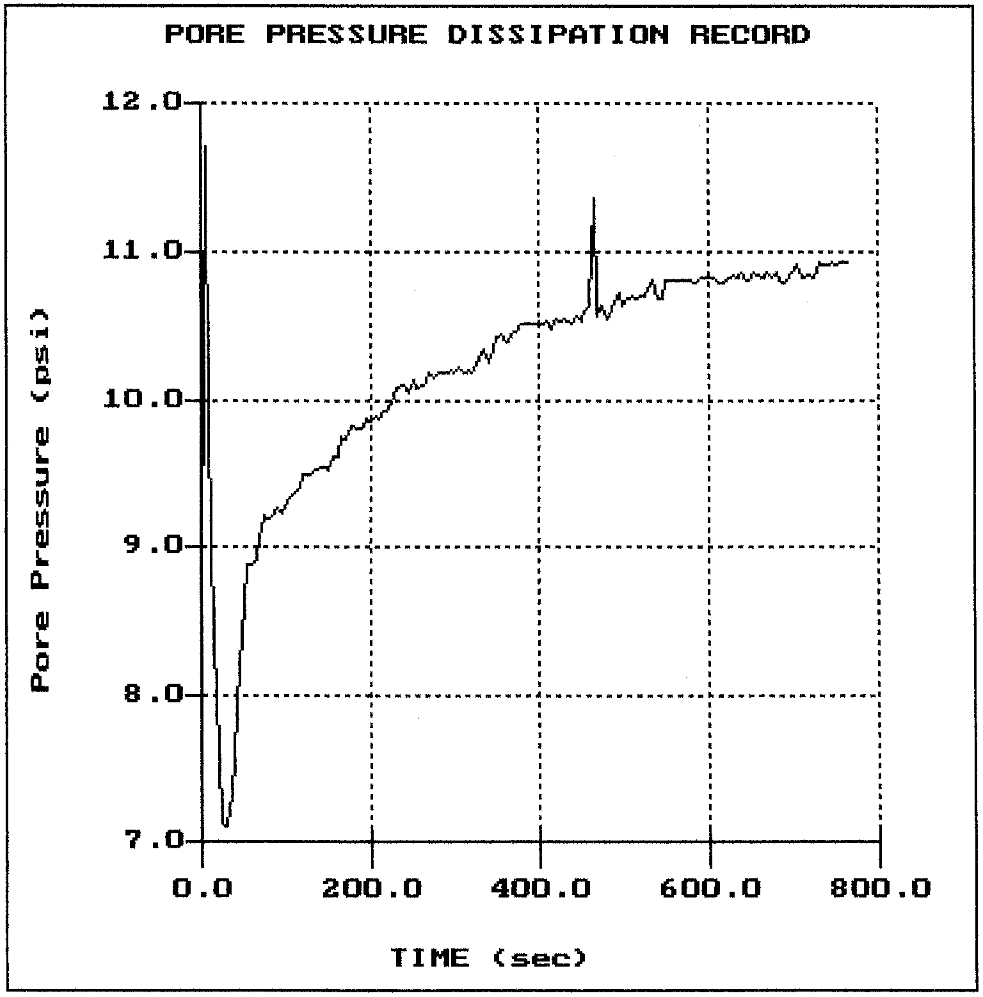
SBT: Soil Behavior Type (Robertson and Campanella 1988)

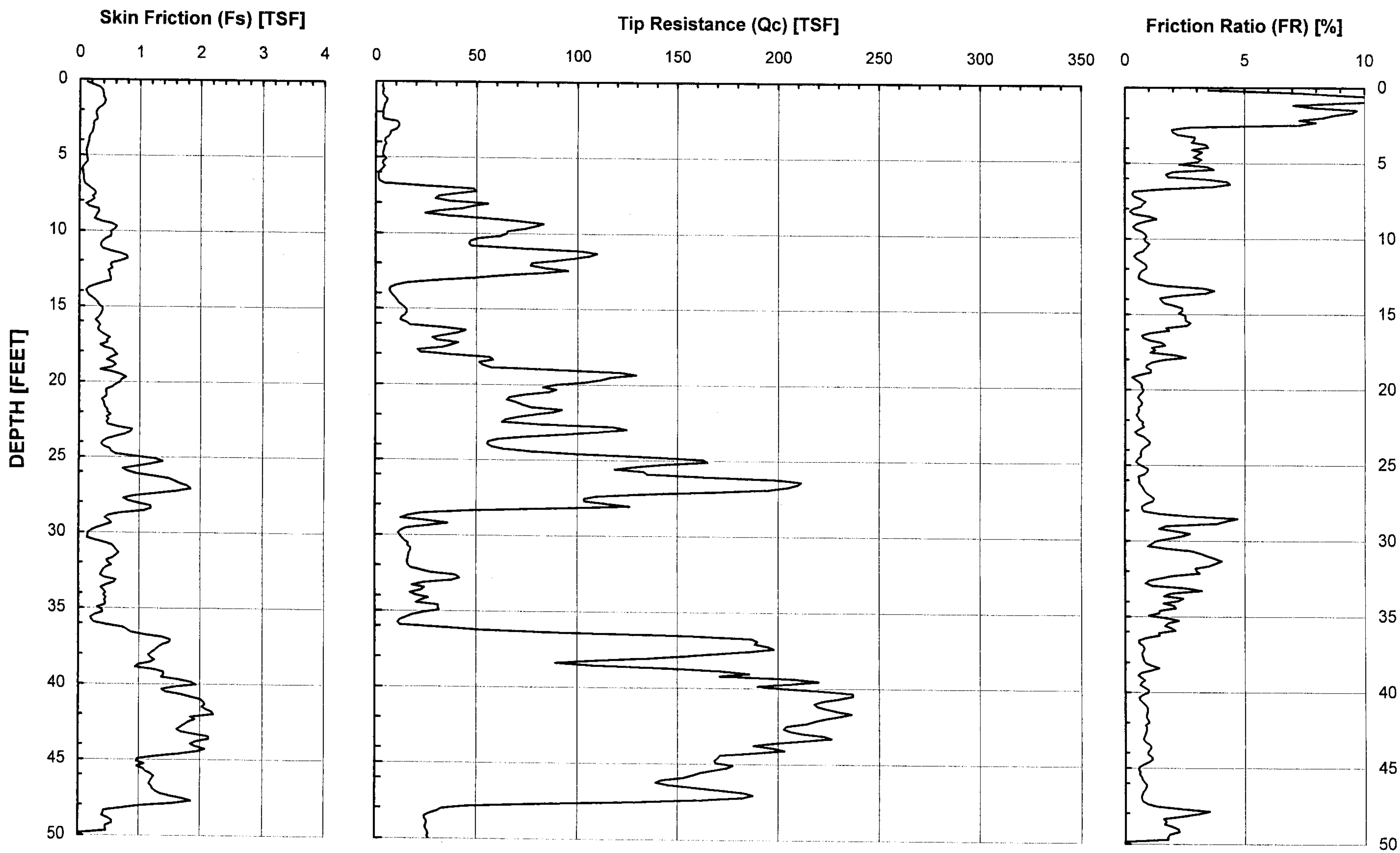
GEOTECHNICS

Site: CAL ENERGY UNIT 6
Location: CPT-06

Engineer: M. EDWARDS
Date:11:28:01 18:55

File: 326C06.PPC
Depth (m): 8.50
(ft): 27.89
Duration : 765.0s
U-min: 7.10 30.0s
U-max: 11.70 5.0s





Geotechnics Incorporated

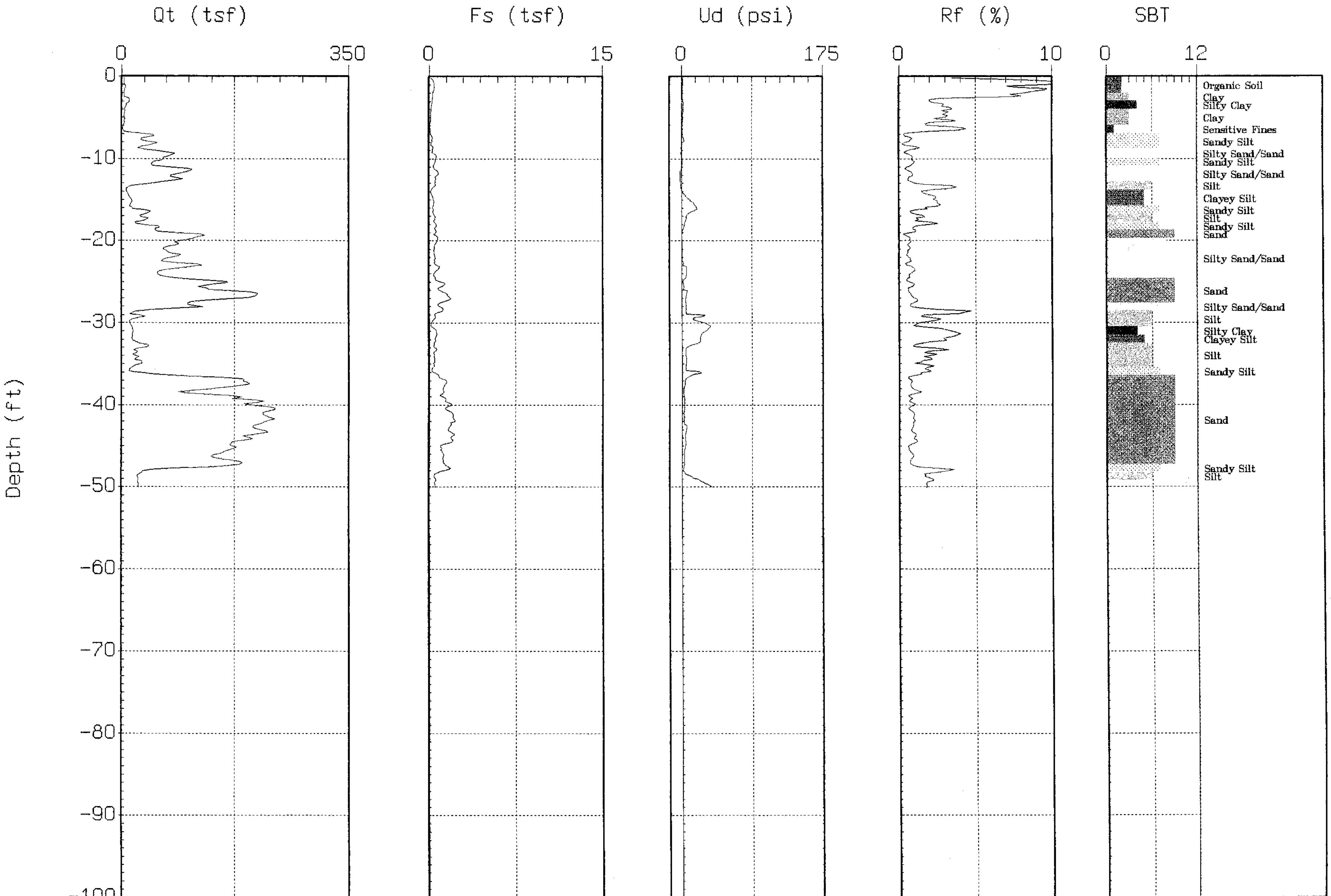
CONE PENETROMETER DATA (CPT-7)

Project No. 0673-002-00
Document No. 02-0022
FIGURE B-7

GREGG GEOTECHNICS

Site : CAL ENERGY UNIT
Location : CPT-07

Engineer : M. EDWARDS
Date : 12:04:01 19:56



Max. Depth: 50.03 (ft)

Depth Inc.: 0.164 (ft)

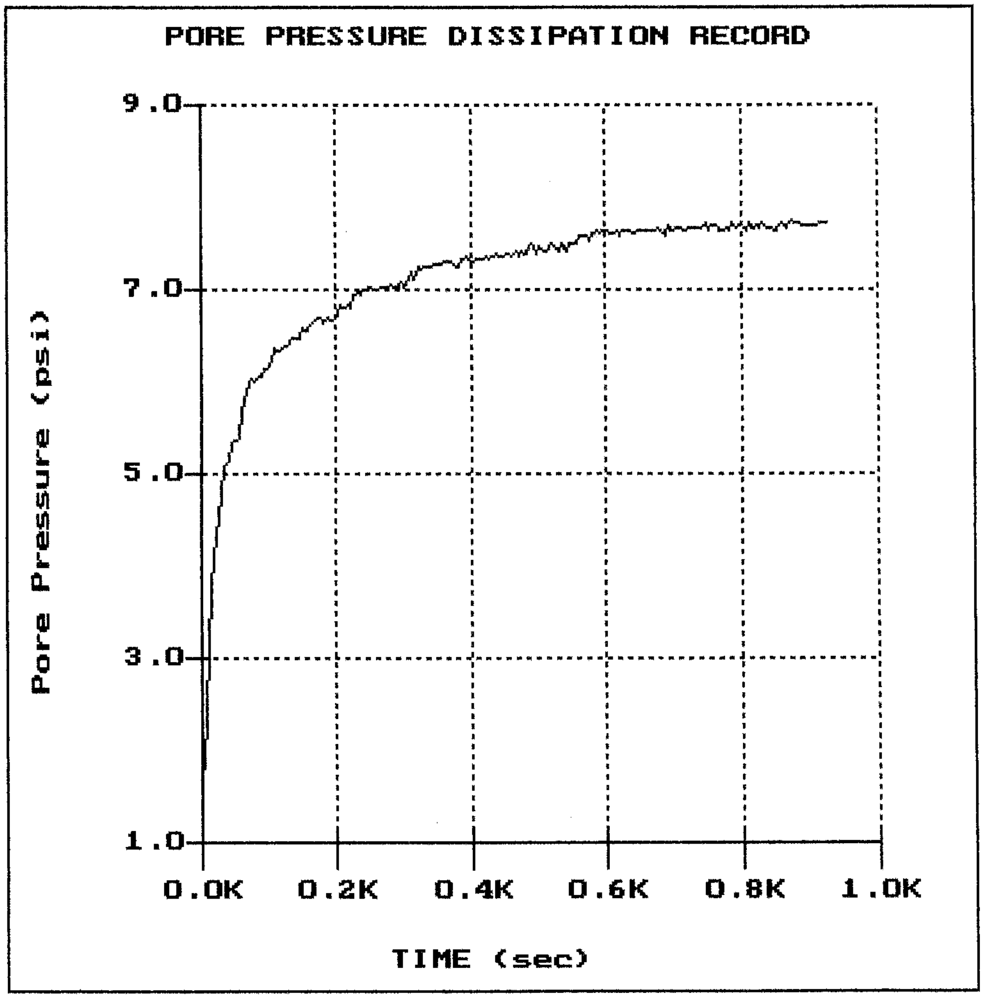
SBT: Soil Behavior Type (Robertson and Campanella 1988)

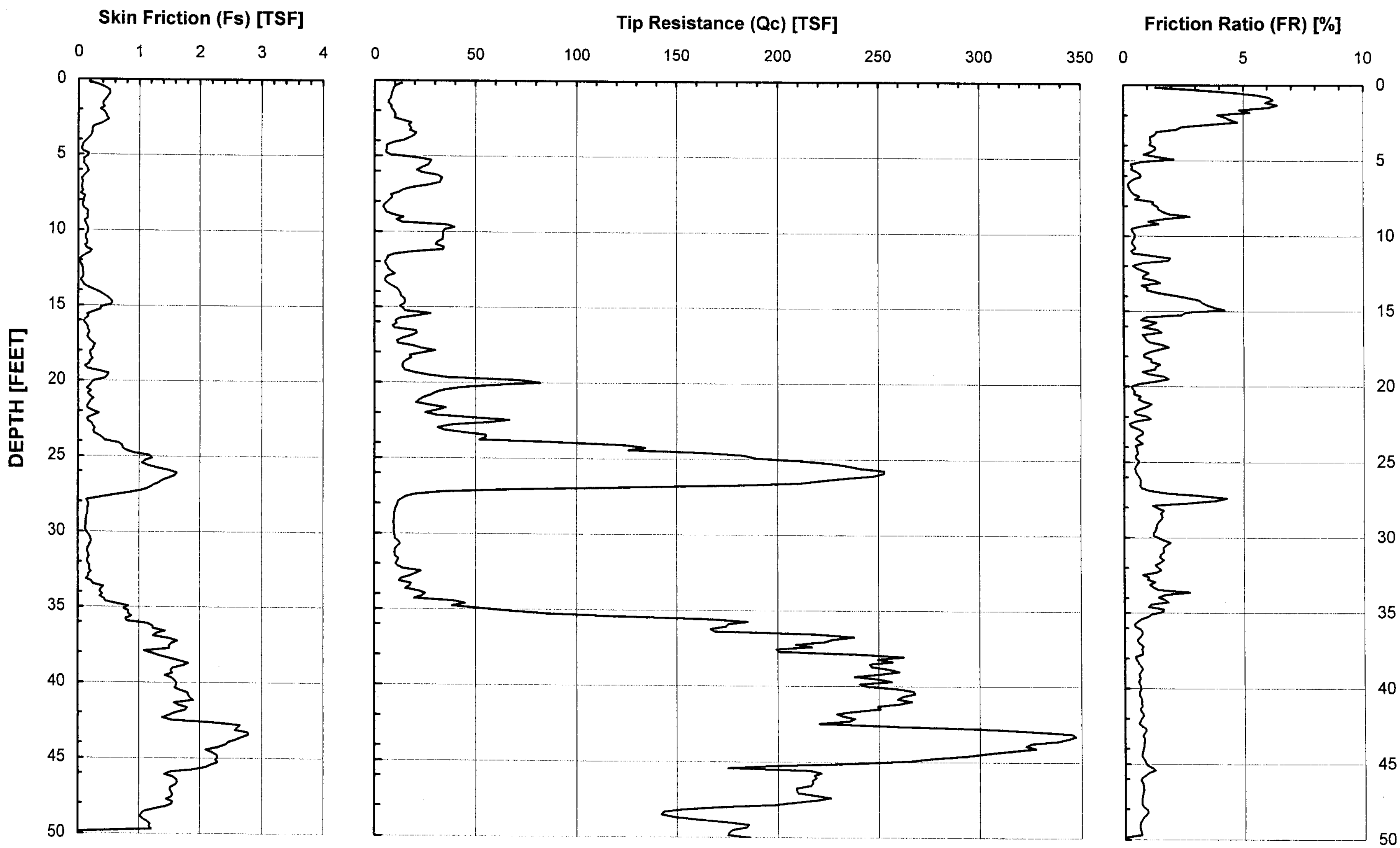
GEOTECHNICS

Site: CAL ENERGY UNIT 6
Location: CPT-07

Engineer: M. EDWARDS
Date:11:28:01 19:56

File: 326C07.PPC
Depth (m): 7.90
(ft): 25.92
Duration : 925.0s
U-min: 1.81 5.0s
U-max: 7.75 875.0s





Geotechnics Incorporated

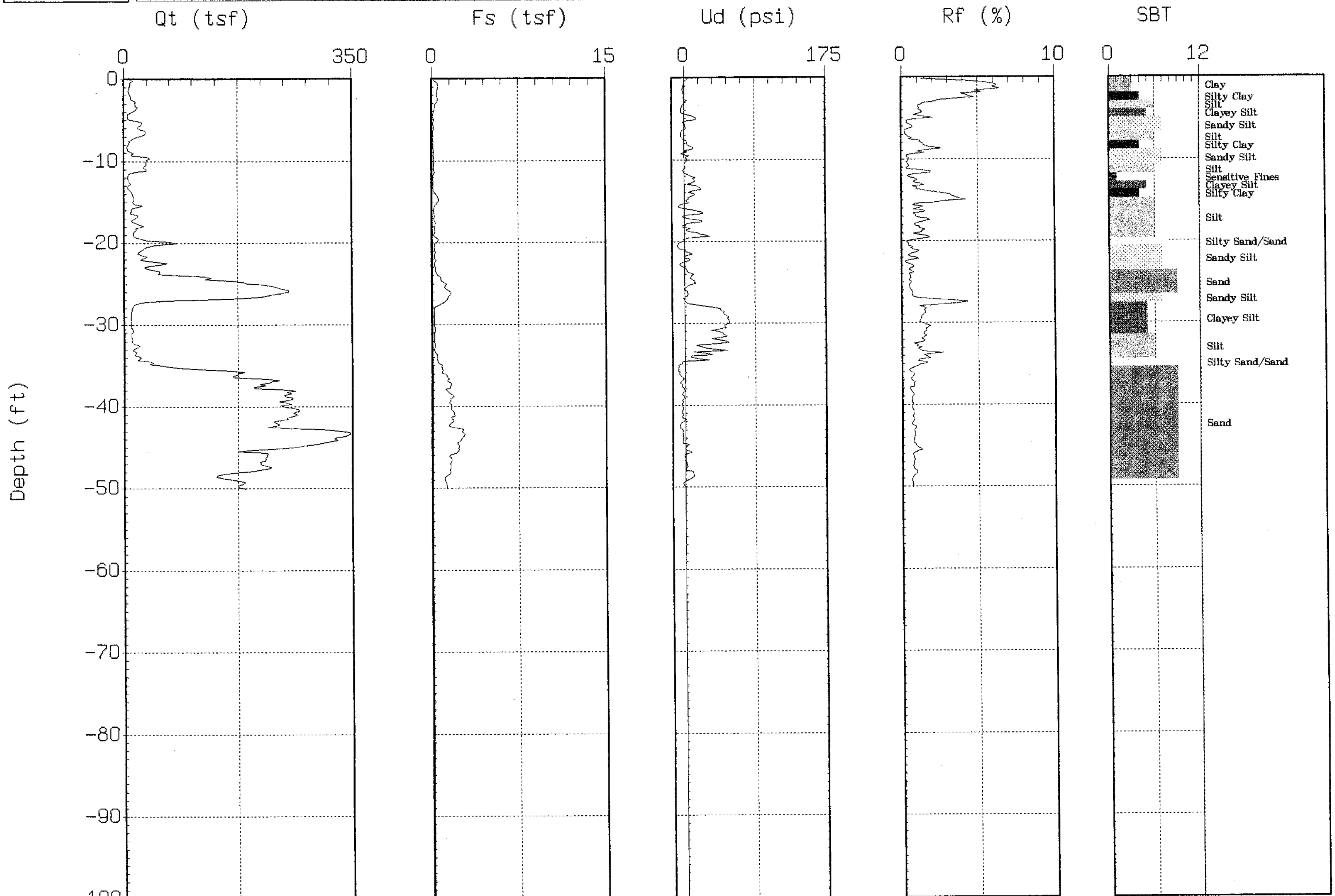
CONE PENETROMETER DATA (CPT-8)

Project No. 0673-002-00
Document No. 02-0022
FIGURE B-8

GREGG GEOTECHNICS

Site : CAL ENERGY UNIT
Location : CPT-08

Engineer : M. EDWARDS
Date : 11:28:01 12:51



Max. Depth: 50.03 (ft)

Depth Inc.: 0.164 (ft)

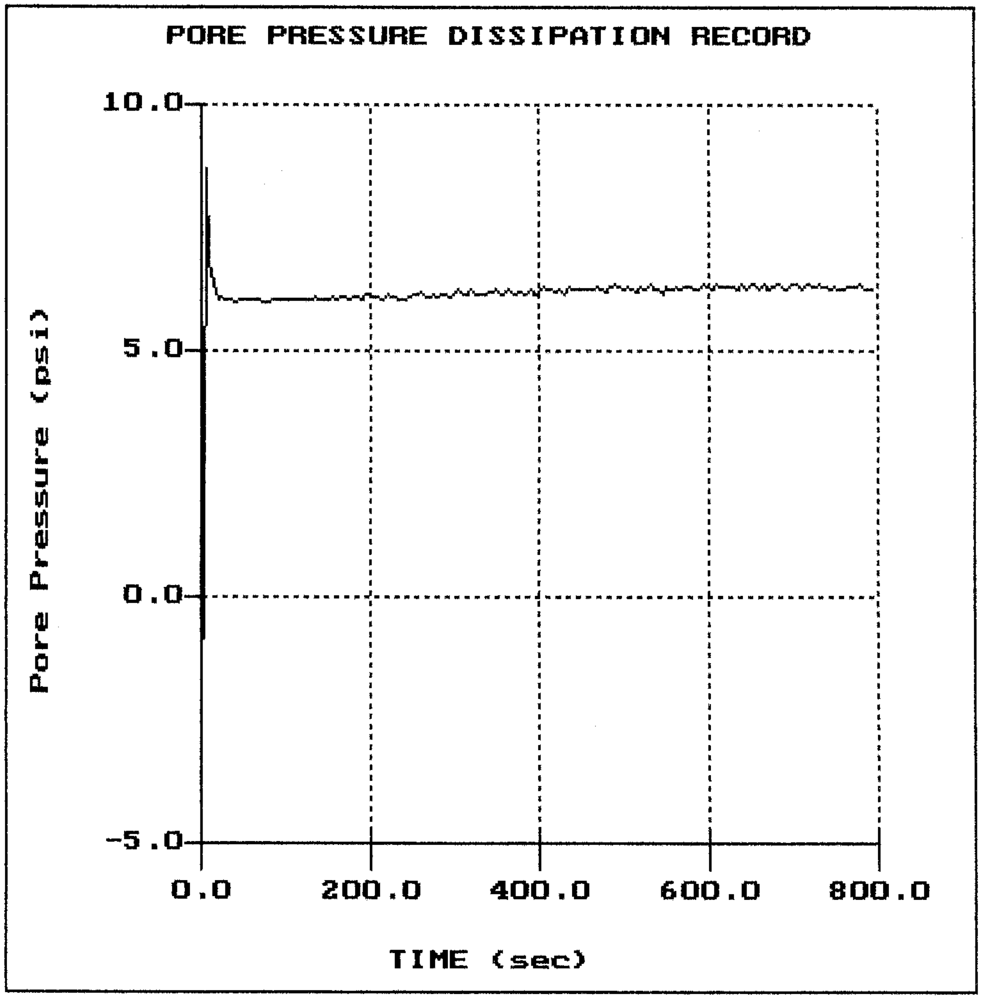
SBT: Soil Behavior Type (Robertson and Campanella 1988)

GEOTECHNICS

Site: CAL ENERGY UNIT 6
Location: CPT-08

Engineer: M. EDWARDS
Date:11:28:01 12:51

File: 326C08.PPC
Depth (m): 6.35
(ft): 20.83
Duration : 795.0s
U-min: -4.05 0.0s
U-max: 8.71 5.0s

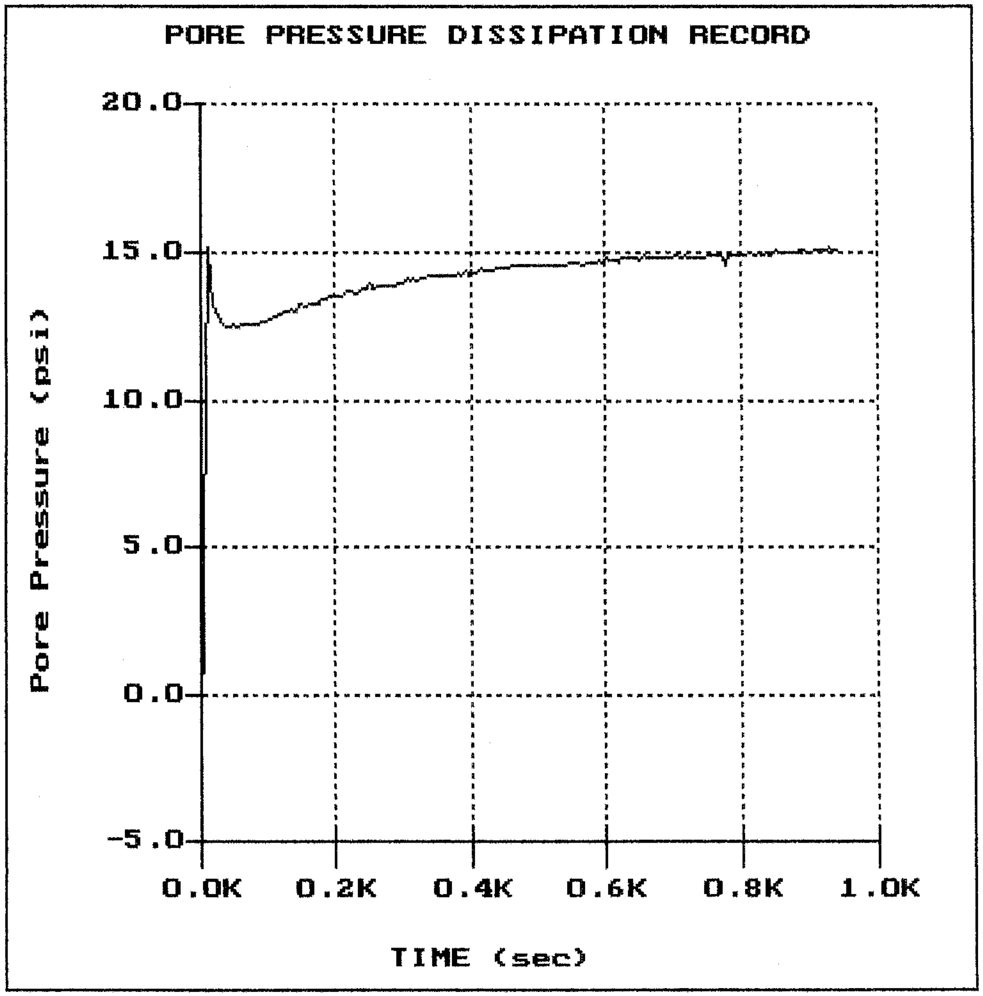


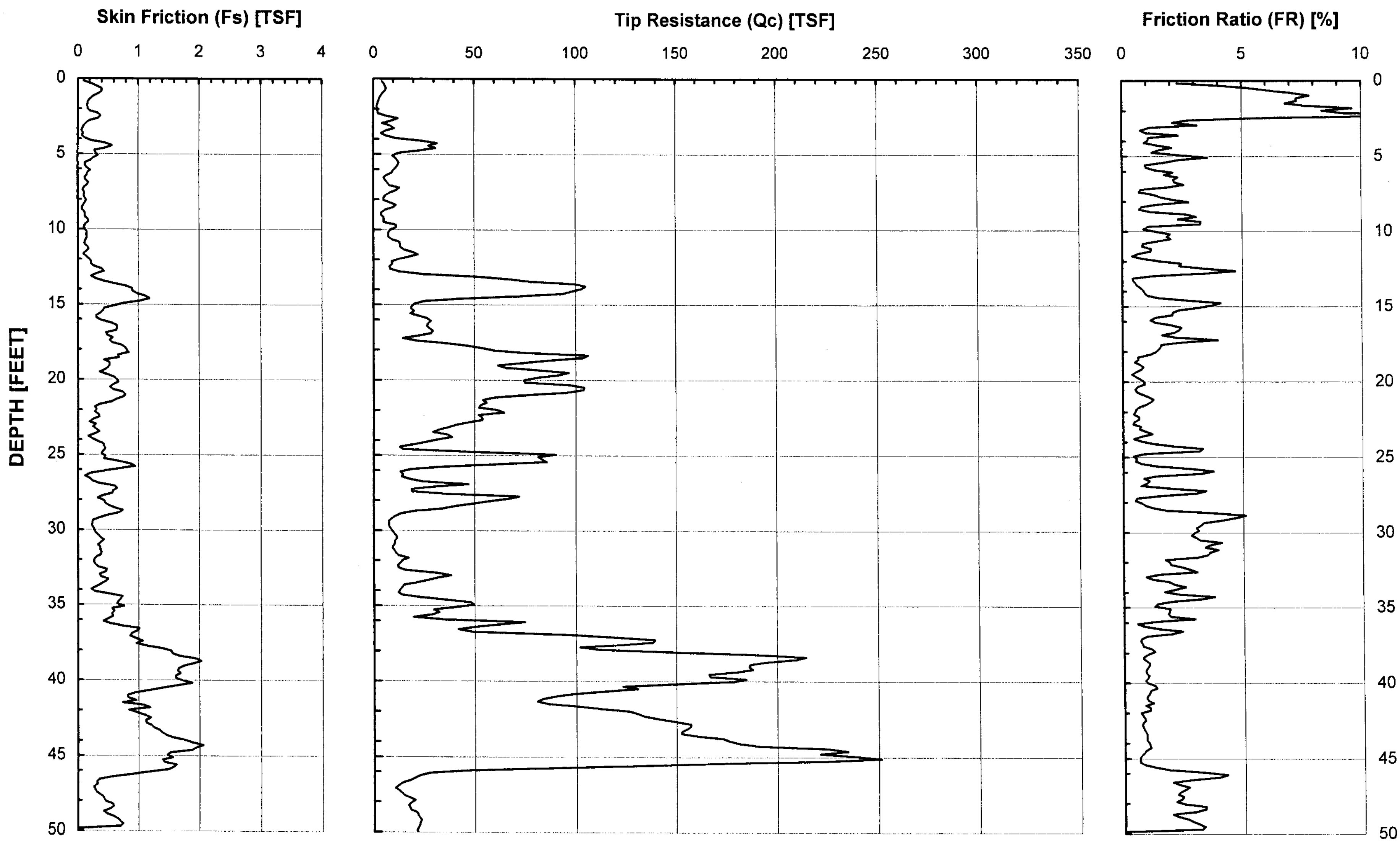
GEOTECHNICS

Site: CAL ENERGY UNIT 6
Location: CPT-08

Engineer: M. EDWARDS
Date:11:28:01 12:51

File: 326C08.PPC
Depth (m): 12.65
(ft): 41.50
Duration : 940.0s
U-min: -3.60 0.0s
U-max: 15.16 930.0s





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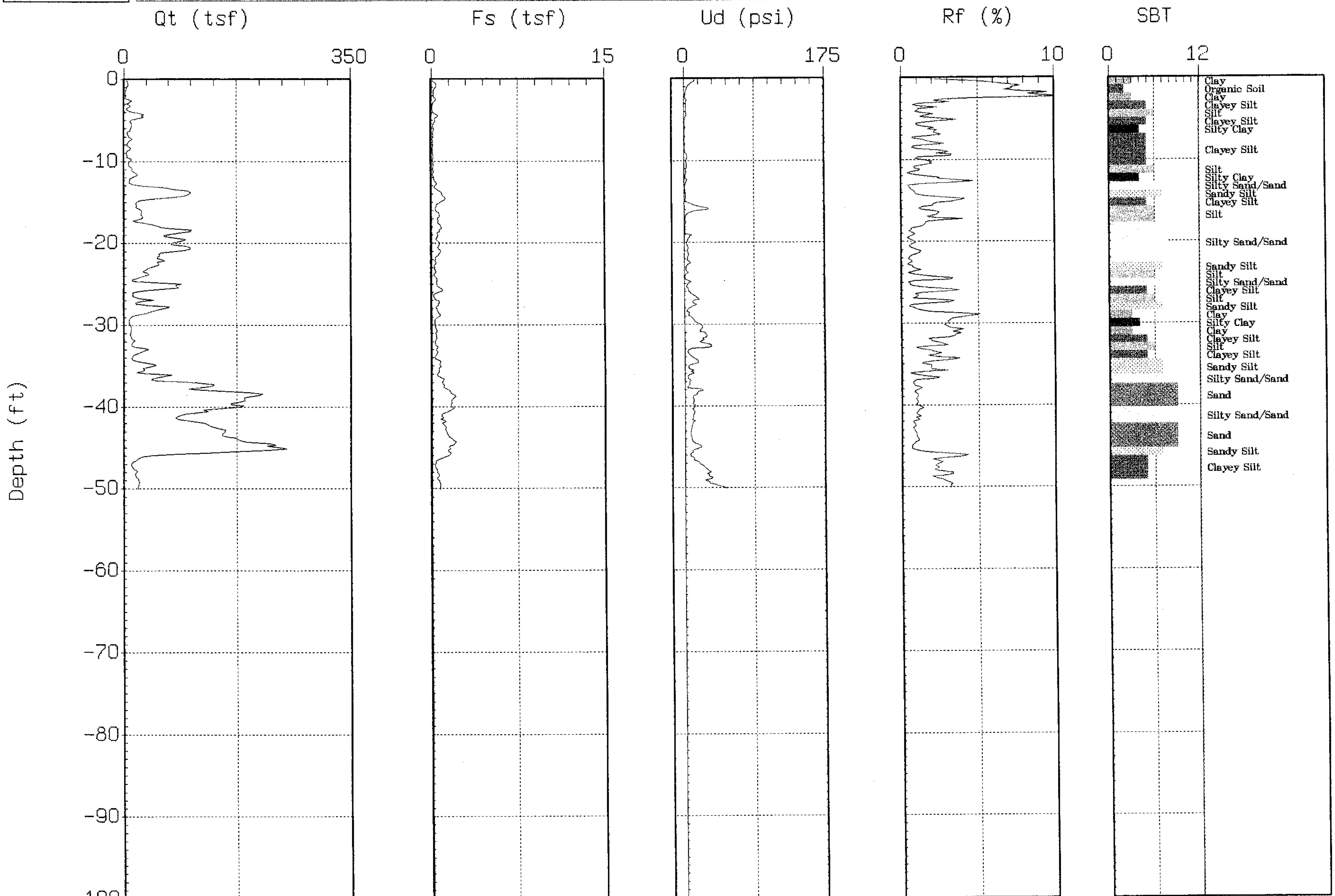
CONE PENETROMETER DATA (CPT-9)

Project No. 0673-002-00
Document No. 02-0022
FIGURE B-9

GREGG GEOTECHNICS

Site : CAL ENERGY UNIT
Location : CPT-09

Engineer : M. EDWARDS
Date : 12:04:01 20:48



Max. Depth: 50.03 (ft)

Depth Inc.: 0.164 (ft)

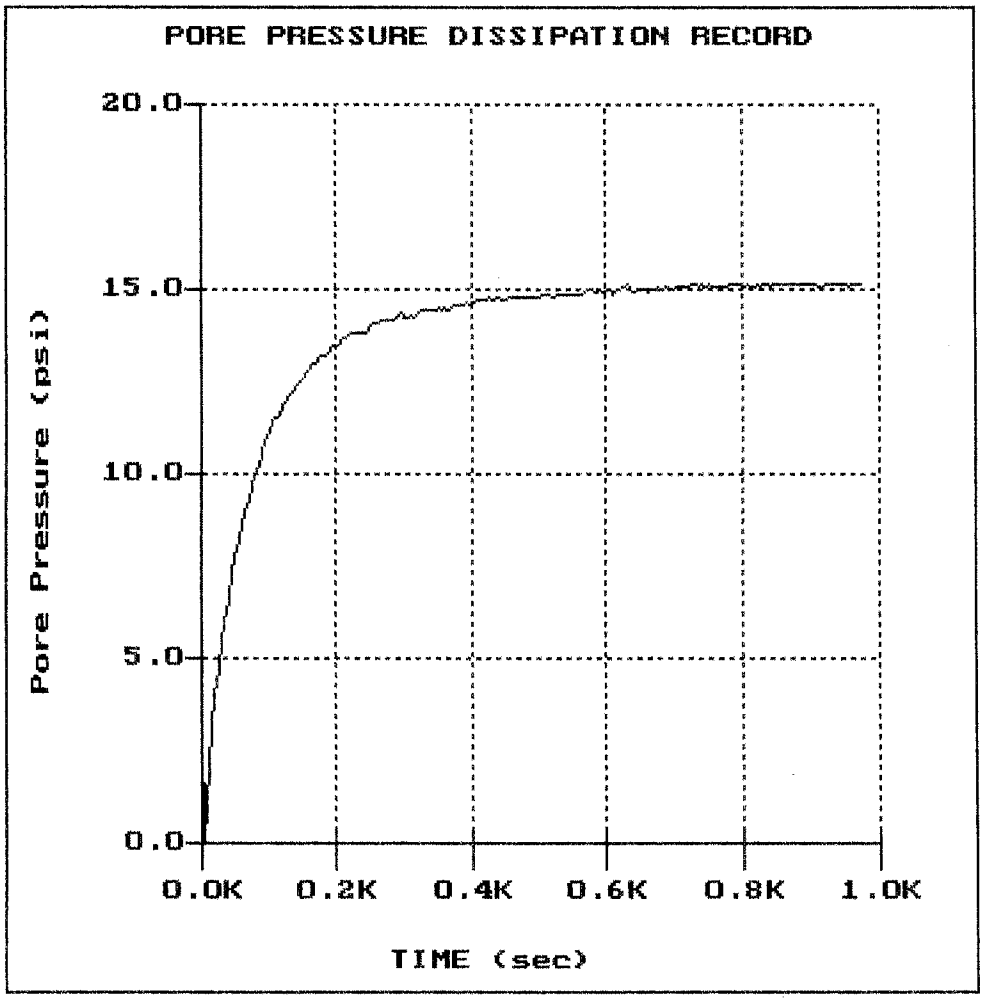
SBT: Soil Behavior Type (Robertson and Campanella 1988)

GEOTECHNICS

Site: CAL ENERGY UNIT 6
Location: CPT-09

Engineer: M. EDWARDS
Date:11:28:01 20:48

File: 326C09.PPC
Depth (m): 11.50
(ft): 37.73
Duration : 975.0s
U-min: -0.22 5.0s
U-max: 15.16 940.0s



LOG OF EXPLORATION BORING NO. B-1

Logged by: DP **Date:** 11/30/01
Method of Drilling: 8-INCH HOLLOW STEM FLIGHT AUGER **Elevation:** - 230' MSL

DEPTH (FEET)	BLOWS PER FT.	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE %	DESCRIPTION	LAB TESTS
1						**LACUSTRINE DEPOSITS**: Lean clay (CL), brown, medium plasticity, moist, soft to firm.	
2							
3							
4							
5				95	26	Silt (ML), brown, nonplastic, saturated, firm.	Gradation Hydrometer Atterberg Sulfate pH & Resistivity Chloride Direct Shear
6							
7							
8							
9							
10	7						
11							
12							
13							
14							
15	4						
16						Interbedded clay (CL), brown, medium plasticity, saturated, firm.	
17							
18							
19							
20				95	29		Gradation
21							
22							
23							
24							
25				97	26		
26							
27							
28							
29							
30						Lean clay (CL), brown, medium plasticity, saturated, firm to hard.	

LOG OF EXPLORATION BORING NO. B-1 (continued)

Logged by: DP **Date:** 11/30/01
Method of Drilling: 8-INCH HOLLOW STEM FLIGHT AUGER **Elevation:** - 230' MSL

DEPTH (FEET)	BLOWS PER FT.	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE %	DESCRIPTION	LAB TESTS
31				96	25	**<u>LACUSTRINE DEPOSITS</u> (continued)**: Lean clay (CL), brown, medium plasticity, saturated, firm to hard.	
32							
33							
34							
35							
36				106	27		
37							
38						Silty sand (SM), brown, fine grained, low plasticity, saturated, medium dense.	
39							
40							
41	12						Gradation
42							
43							
44							
45						Lean clay (CL), brown, medium plasticity, saturated, soft to firm.	
46	4						
47							
48							
49							
50							
51				84	37		
52							
53							
54							
55							
56				96	27		
57							
58							
59							
60						Brown silty sand (SM), fine grained, low plasticity, saturated, medium dense.	

LOG OF EXPLORATION BORING NO. B-1 (continued)

Logged by: DP **Date:** 11/30/01
Method of Drilling: 8-INCH HOLLOW STEM FLIGHT AUGER **Elevation:** - 230' MSL

DEPTH (FEET)	BLOWS PER FT.	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE %	DESCRIPTION	LAB TESTS
				103	23		
61						**<u>LACUSTRINE DEPOSITS</u> (continued)**: Silty sand (SM), brown, low plasticity, saturated, medium dense.	Gradation
62							
63							
64						Silt (ML), brown, low plasticity, saturated, firm to hard.	
65	8						
66							
67							
68							
69							
70				101	26		
71							
72							
73							
74							
75				103	24		
76							
77							
78						Total Depth = 77½ feet Groundwater encountered at @ 4 feet Backfilled 11/30/01	
79							
80							
81							
82							
83							
84							
85							
86							
87							
88							
89							
90							

LOG OF EXPLORATION BORING NO. B- 2

Logged by: ME **Date:** 11/29/01

Method of Drilling: 8-INCH HOLLOW STEM FLIGHT AUGER **Elevation:** - 230' MSL

DEPTH (FEET)	BLOWS PER FT.	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE %	DESCRIPTION	LAB TESTS
1						**LACUSTRINE DEPOSITS**: Lean clay (CL), brown, medium plasticity, moist, soft to firm.	
2							
3							Gradation Hydrometer Atterberg Sulfate pH & Resistivity Chloride Expansion R-Value
4							
5						Interbedded lean clay (CL), brown, medium plasticity, saturated, soft to firm and silt (ML), brown, nonplastic, saturated, soft.	
6							
7							
8							
9							
10				95	30		Gradation Consolidation Direct Shear
11							
12							
13							
14							
15				95	29		Gradation Consolidation
16							
17							
18							
19							
20				92	31		Gradation Consolidation
21							
22							
23							
24							
25				92	23		Gradation Consolidation
26							
27						Clayey sand (SC), brown, fine to medium grained, low plasticity, saturated, medium dense.	
28							
29							
30						Lean clay (CL), brown, medium plasticity, saturated, soft to firm.	

PROJECT NO. 0673-002-00 **GEOTECHNICS INCORPORATED** **FIGURE: B-13**

LOG OF EXPLORATION BORING NO. B-2 (continued)

Logged by: ME **Date:** 11/29/01
Method of Drilling: 8-INCH HOLLOW STEM FLIGHT AUGER **Elevation:** - 230' MSL

DEPTH (FEET)	BLOWS PER FT.	DRIVE SAMPLE	BULK SAMPLE	DENSITY (PCF)	MOISTURE %	DESCRIPTION	LAB TESTS
31	2					**LACUSTRINE DEPOSITS (continued)**: Lean clay (CL), brown, medium plasticity, saturated, firm to hard.	
32							
33							
34							
35							
36				102	24		
37							
38							
39							
40							
41				95	29		
42							
43							
44							
45							
46					26		
47							
48							
49						Clayey sand (SC), brown, fine to medium grained, low plasticity, saturated, medium dense.	
50							
51	21						
52							
53							
54							
55							
56							
57							
58							
59						Total Depth = 58 feet	
60						Groundwater encountered at @ 4 feet Backfilled 11/30/01	

APPENDIX C

LABORATORY TESTING

Laboratory testing was conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions and in the same locality. No other warranty, expressed or implied, is made as to the correctness or serviceability of the test results or the conclusions derived from these tests. Where a specific laboratory test method has been referenced, such as ASTM, Caltrans, or AASHTO, the reference applies only to the specified laboratory test method and not to associated referenced test method(s) or practices, and the test method referenced has been used only as a guidance document for the general performance of the test and not as a "Test Standard". A brief description of the tests performed follows.

Classification: Soils were classified visually according to the Unified Soil Classification System. Visual classification was supplemented by laboratory testing of selected samples and classification in accordance with ASTM D2487. The classifications are shown on the Boring Logs.

In-Situ Moisture/Density: The in-situ moisture contents and dry unit weights of several samples were determined from Shelby Tube samples in general accordance with ASTM test methods D2216 and D2937. The dry unit weights and moisture contents are shown on the Boring Logs.

Particle Size Analysis: Particle size analyses were performed in general accordance with ASTM D422. The grain size distributions were used for the liquefaction analyses and to determine presumptive soil parameters. The results are given in Figures C-1.1 through C-1.9.

Atterberg Limits: ASTM D4318-84 was used to determine the liquid limit, plastic limit, and plasticity index of selected samples. The results are shown in Figures C-1.1 and C-1.5. The abbreviation "NP" is used for nonplastic.

pH and Resistivity: To assess the potential for reactivity with buried metal pipe and below grade ferrous materials, selected soil samples were tested for pH and resistivity in general accordance with the laboratory procedures outlined in Caltrans test method 643. The results are shown in Figure C-2.

Sulfate Content: To assess the potential for reactivity with below grade concrete, selected soil samples were tested for water soluble sulfate content. The water soluble sulfate was extracted under vacuum from the soil using a 10:1 (water to dry soil) dilution ratio (or diluted as necessary and then corrected back to reflect a 10:1 dilution ratio). The extracted solution was then tested for water soluble sulfate in general accordance with ASTM D516. The results are presented in Figure C-2.

APPENDIX C

LABORATORY TESTING (Continued)

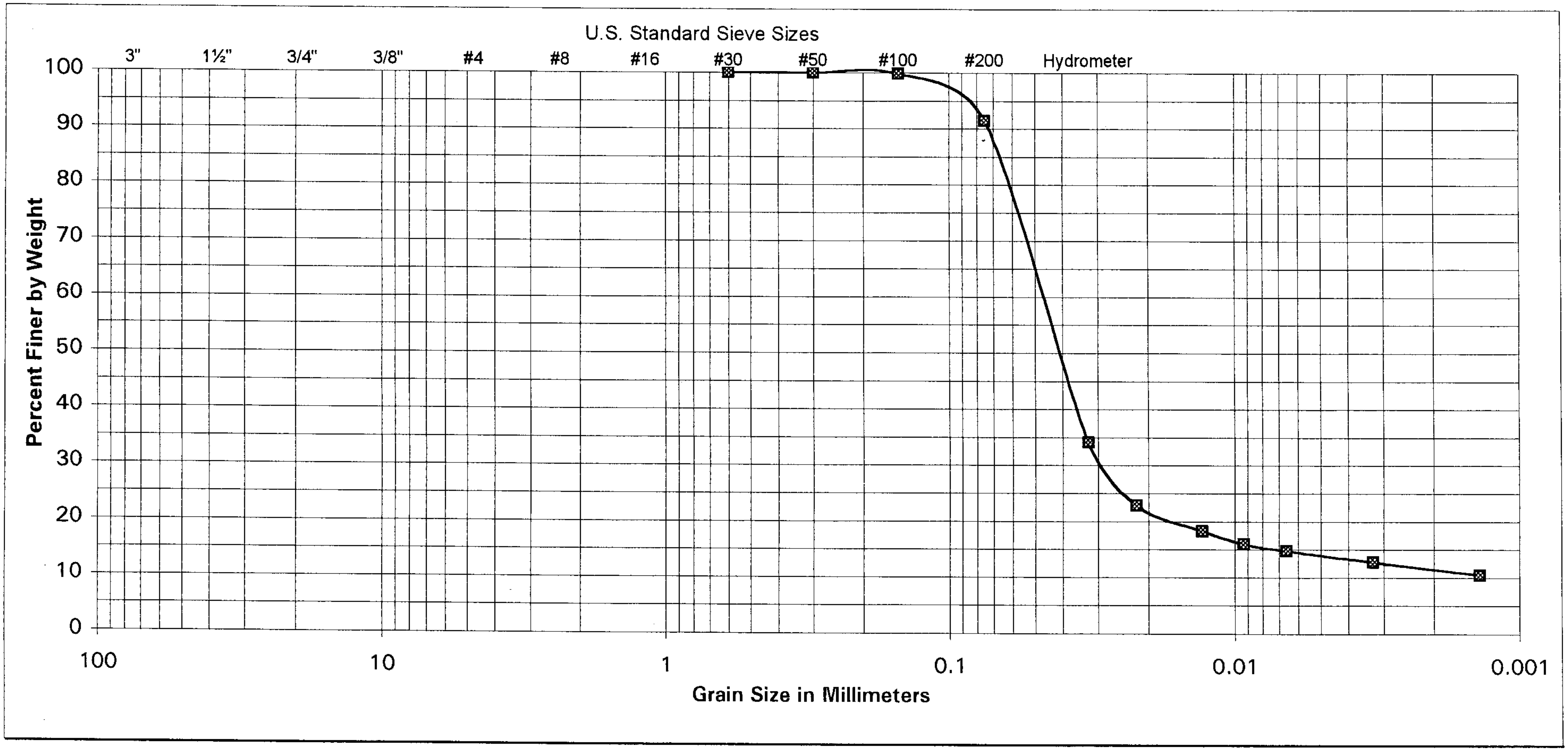
Chloride Content: Selected soil samples were tested for water-soluble chloride content using EPA Test Method SMEWW 4500 CLC. The results are also shown in Figure C-2.

Expansion Index: The expansion potential of selected soils was estimated in general accordance with the laboratory procedures outlined in ASTM test method D4829. Figure C-3 provides the results of these tests.

Direct Shear Test: The shear strength of the selected surficial soils was assessed using direct shear tests performed in general accordance with ASTM D3080. The results are summarized in Figures C-4.1 and C-4.2.

Consolidation Test: In order to evaluate the compressive behavior of the site soils, consolidation tests were conducted in general accordance with ASTM D2435. The samples were given a nominal seating load and saturated prior to loading. The results are shown in Figures C-5.1 through C-5.4.

R-Value: An R-Value test was performed on selected pavement area materials in general accordance with the laboratory procedures outlined in California Test Method 301. The test results are presented in Figures C-6.1 and C-6.2. Pavement calculations for asphalt and base sections corresponding to various traffic levels at the site were conducted in general accordance with the Caltrans Design Method (Topic 608.4). The results are presented in Figures C-6.3 through C-6.6.



GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE	
SAMPLE NUMBER:	B1
SAMPLE LOCATION:	5' - 6'

UNIFIED SOIL CLASSIFICATION: ML

DESCRIPTION: SILT

ATTERBERG LIMITS	
LIQUID LIMIT:	NP
PLASTIC LIMIT:	NP
PLASTICITY INDEX:	NP

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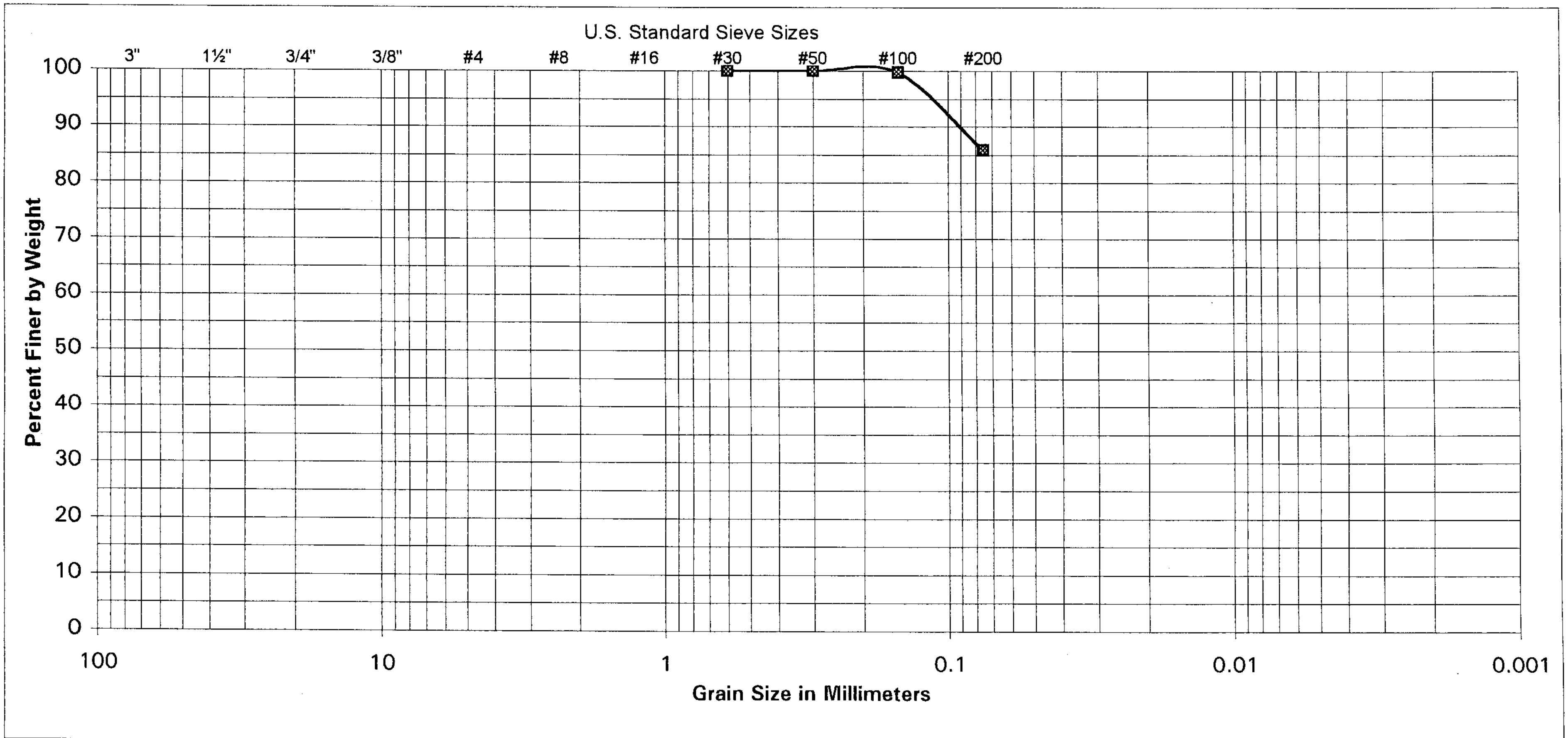
SOIL CLASSIFICATION

Project No. 0673-002-00
Document No. 02-0022
FIGURE C-1.1



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE	
SAMPLE NUMBER:	B1
SAMPLE LOCATION:	20' - 22'

UNIFIED SOIL CLASSIFICATION: ML

DESCRIPTION: SILT WITH SAND

ATTERBERG LIMITS
LIQUID LIMIT:
PLASTIC LIMIT:
PLASTICITY INDEX:

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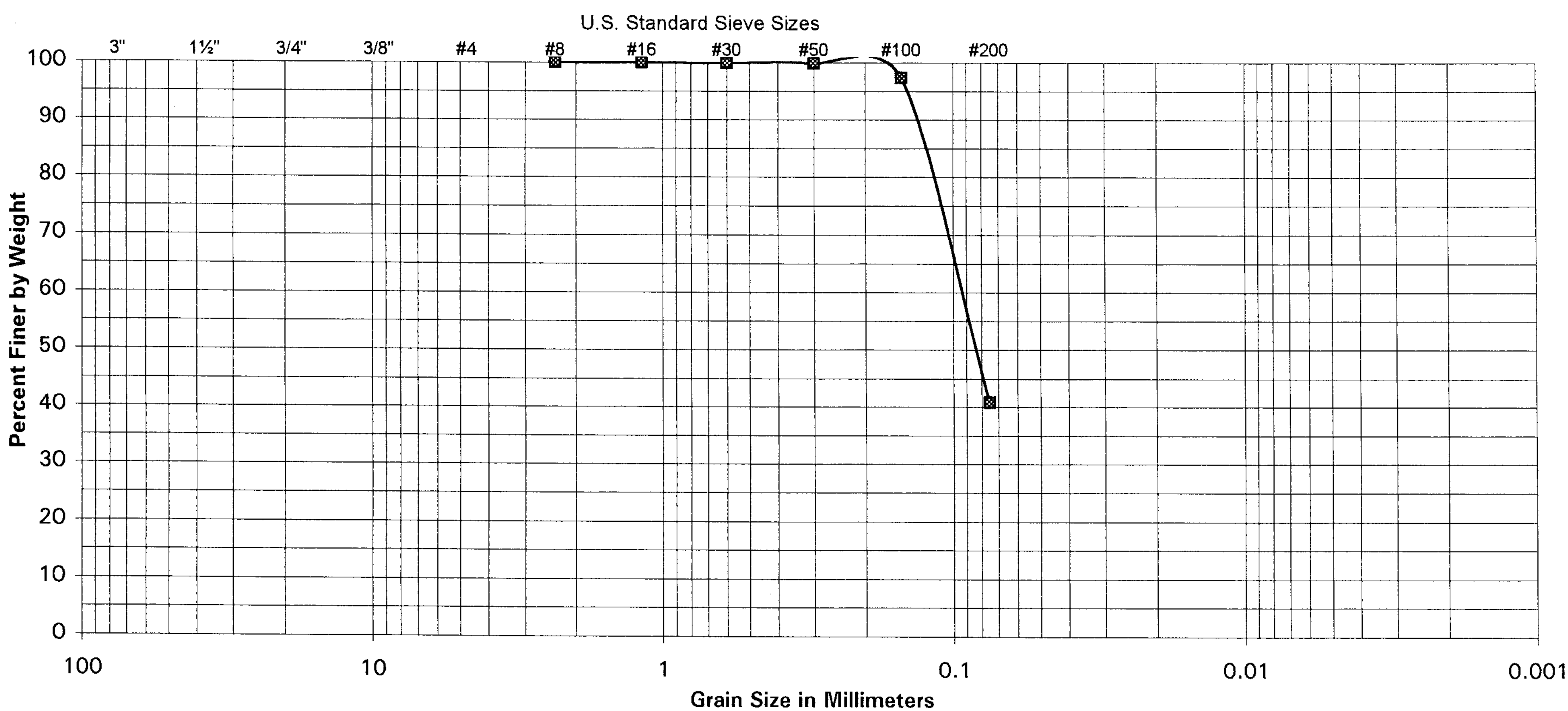
SOIL CLASSIFICATION

Project No. 0673-002-00
Document No. 02-0022
FIGURE C-1.2



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE	
SAMPLE NUMBER:	B1
SAMPLE LOCATION:	40' - 42'

UNIFIED SOIL CLASSIFICATION: SM

DESCRIPTION: SILTY SAND

ATTERBERG LIMITS
LIQUID LIMIT:
PLASTIC LIMIT:
PLASTICITY INDEX:

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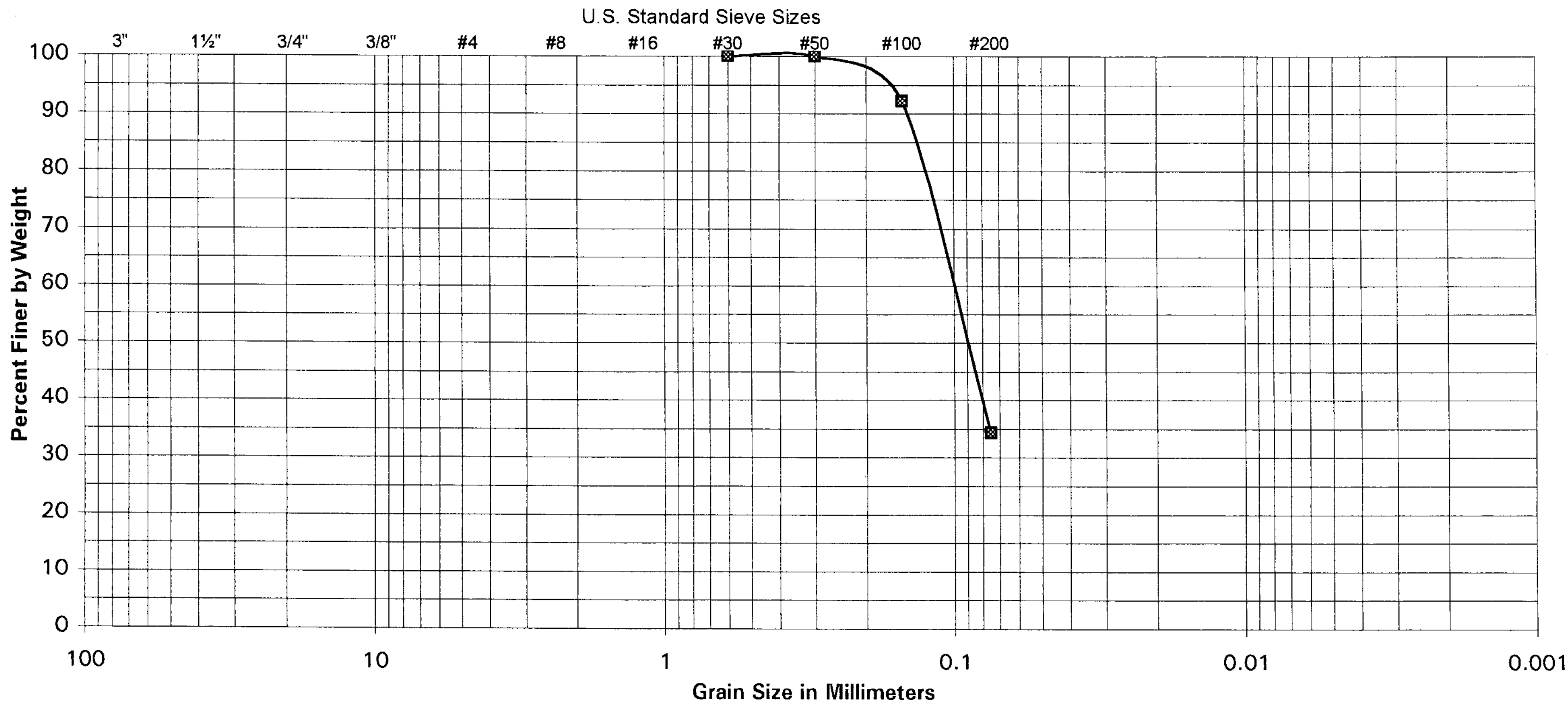
SOIL CLASSIFICATION

Project No. 0673-002-00
Document No. 02-0022
FIGURE C-1.3



GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE	
SAMPLE NUMBER:	B1
SAMPLE LOCATION:	60' - 62'

UNIFIED SOIL CLASSIFICATION: SM

DESCRIPTION: SILTY SAND

ATTERBERG LIMITS
LIQUID LIMIT:
PLASTIC LIMIT:
PLASTICITY INDEX:

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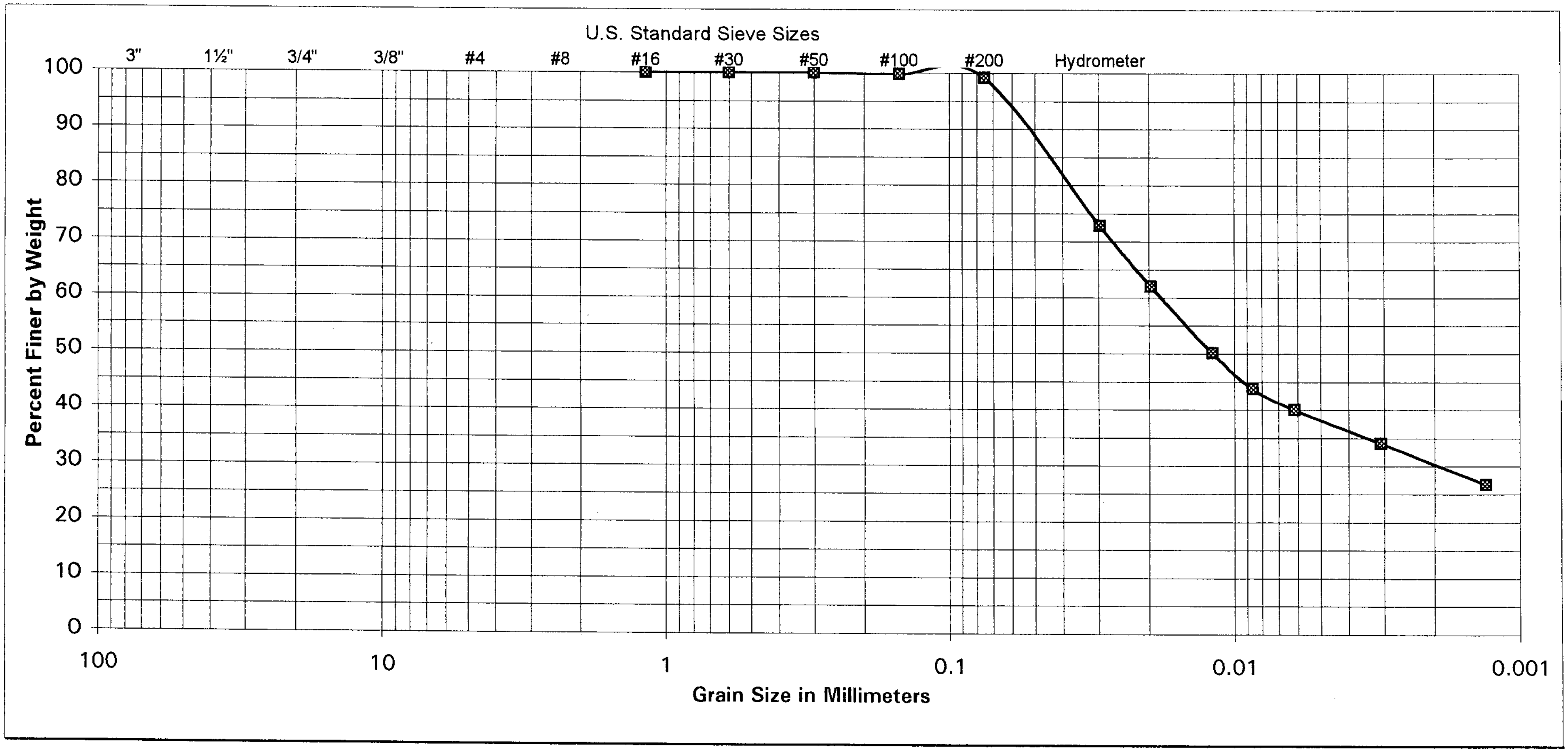
SOIL CLASSIFICATION

Project No. 0673-002-00
Document No. 02-0022
FIGURE C-1.4



GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE	
SAMPLE NUMBER:	B2
SAMPLE LOCATION:	2' - 4'

UNIFIED SOIL CLASSIFICATION: CL

DESCRIPTION: LEAN CLAY

ATTERBERG LIMITS	
LIQUID LIMIT:	40
PLASTIC LIMIT:	20
PLASTICITY INDEX:	20

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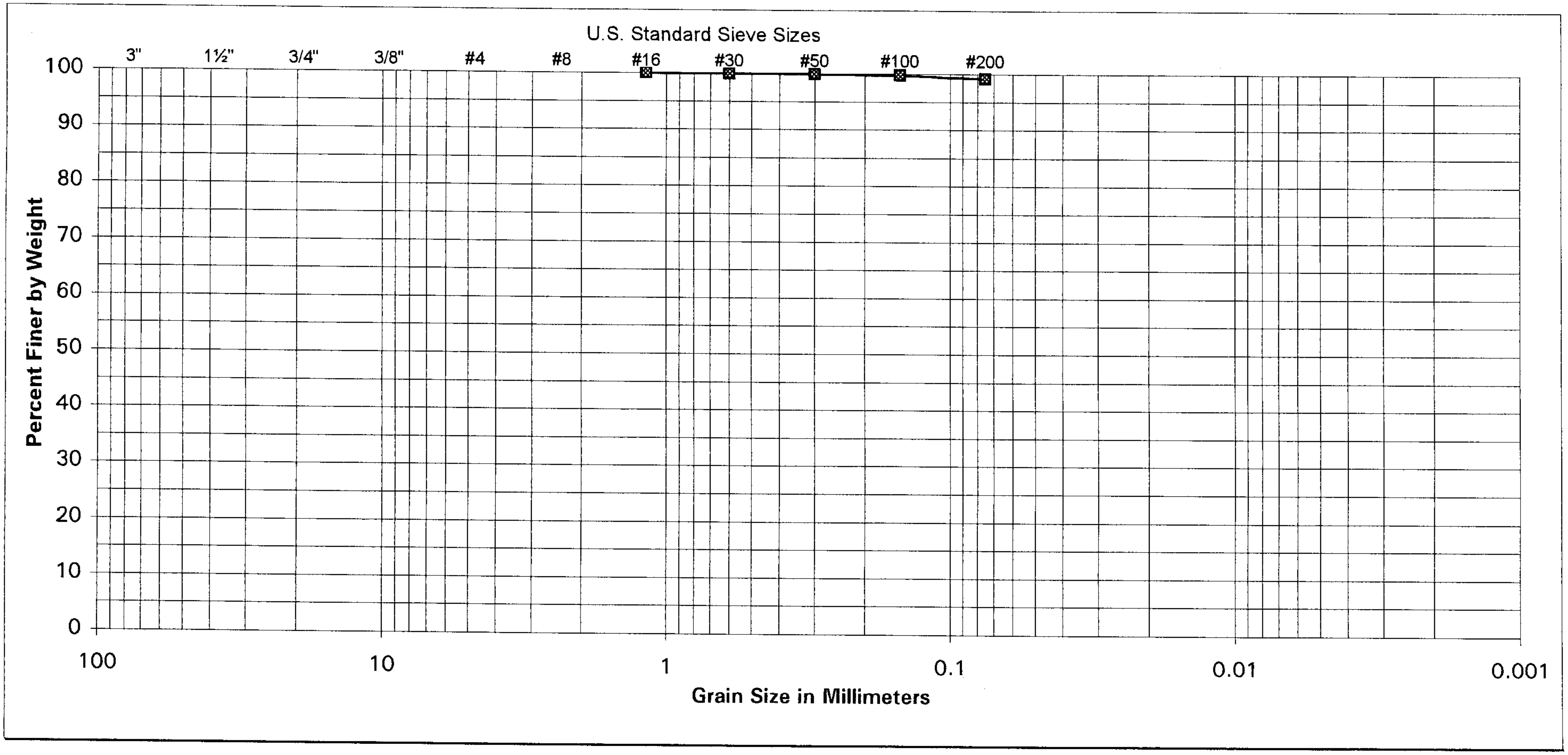
SOIL CLASSIFICATION

Project No. 0673-002-00
Document No. 02-0022
FIGURE C-1.5



GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE	
SAMPLE NUMBER:	B2
SAMPLE LOCATION:	10' - 12'

UNIFIED SOIL CLASSIFICATION: ML

DESCRIPTION: SILT

ATTERBERG LIMITS	
LIQUID LIMIT:	
PLASTIC LIMIT:	
PLASTICITY INDEX:	

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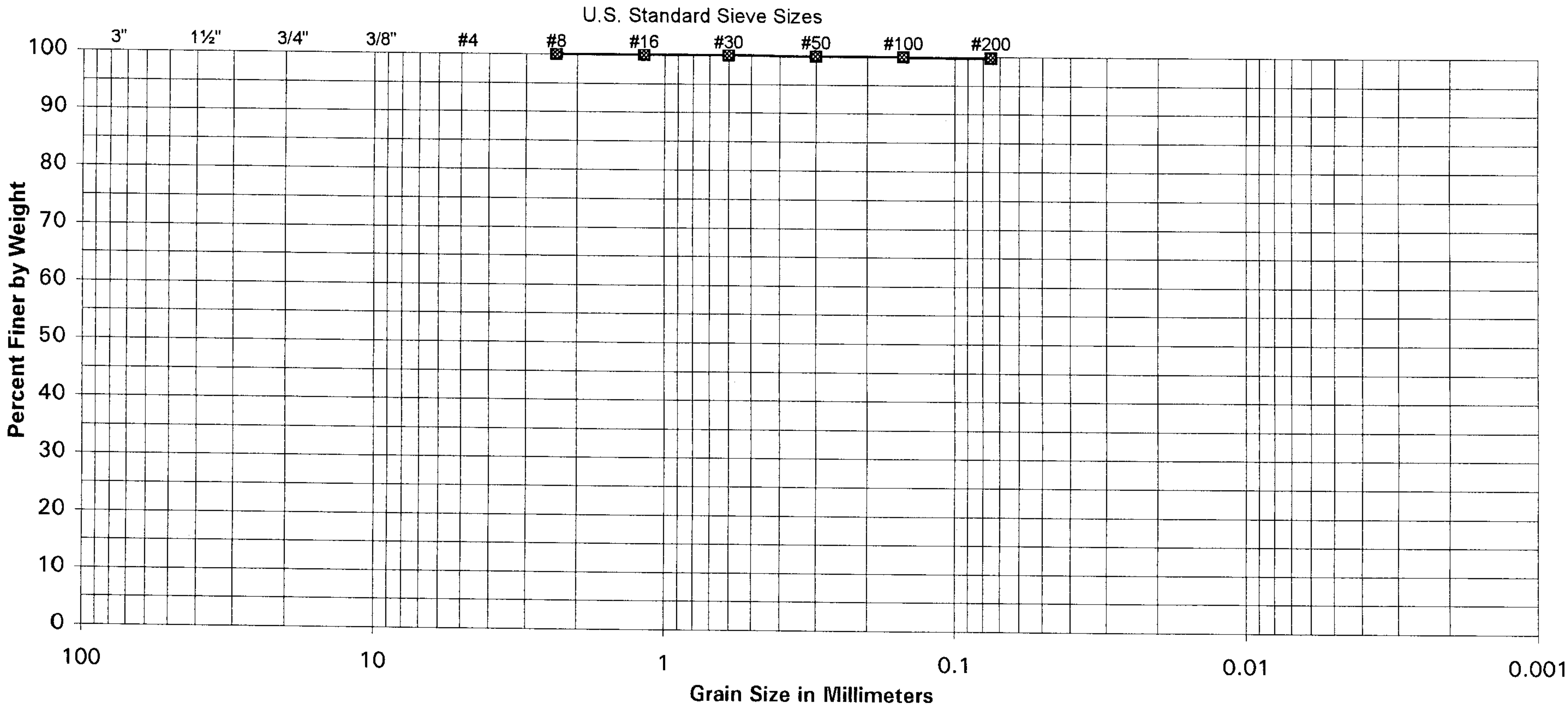
SOIL CLASSIFICATION

Project No. 0673-002-00
Document No. 02-0022
FIGURE C-1.6



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE

SAMPLE NUMBER: B2

SAMPLE LOCATION: 15' - 17'

UNIFIED SOIL CLASSIFICATION: CL

DESCRIPTION: LEAN CLAY

ATTERBERG LIMITS

LIQUID LIMIT:

PLASTIC LIMIT:

PLASTICITY INDEX:

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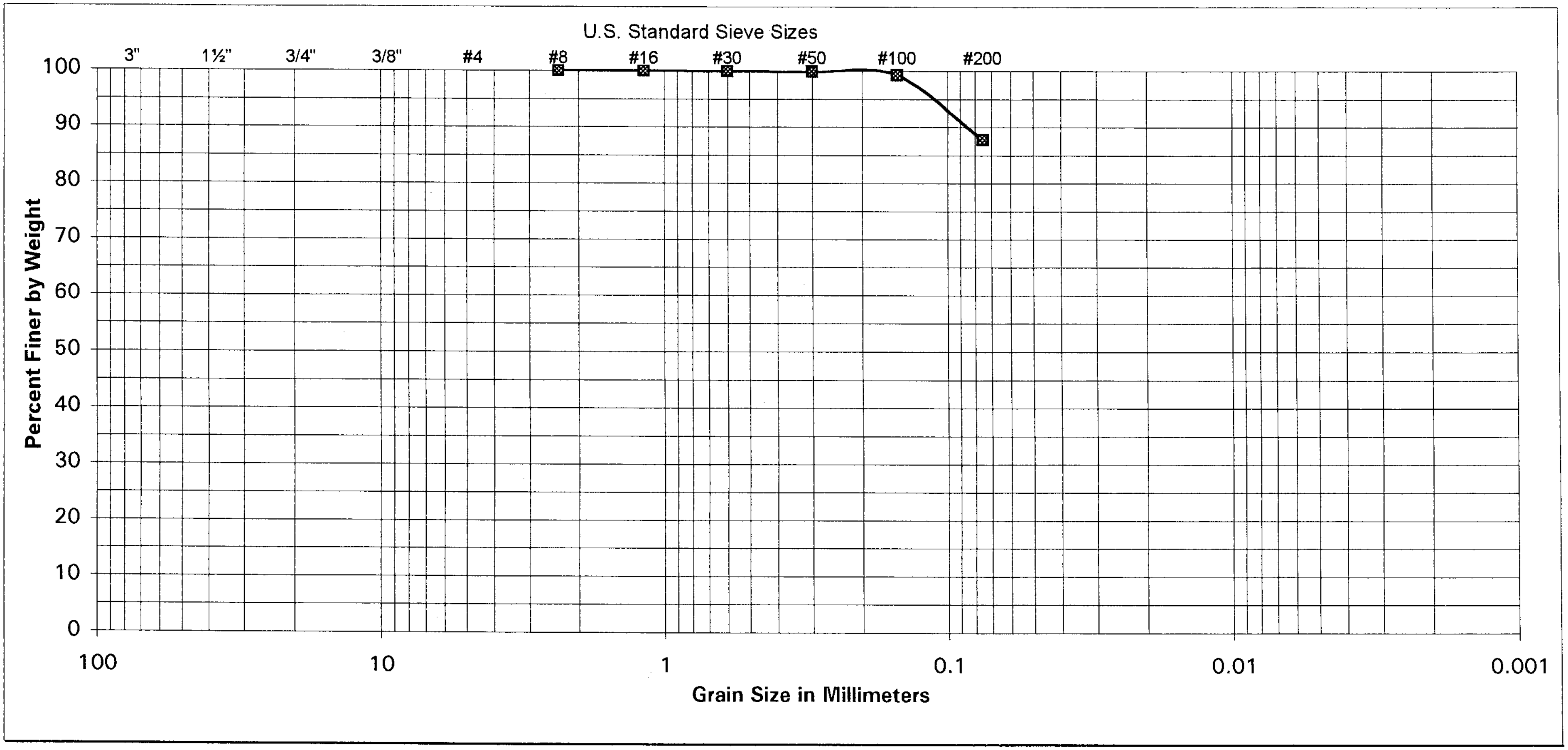
SOIL CLASSIFICATION

Project No. 0673-002-00

Document No. 02-0022

FIGURE C-1.7



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
SAMPLE NUMBER:	B2
SAMPLE LOCATION:	20' - 22'

UNIFIED SOIL CLASSIFICATION: ML

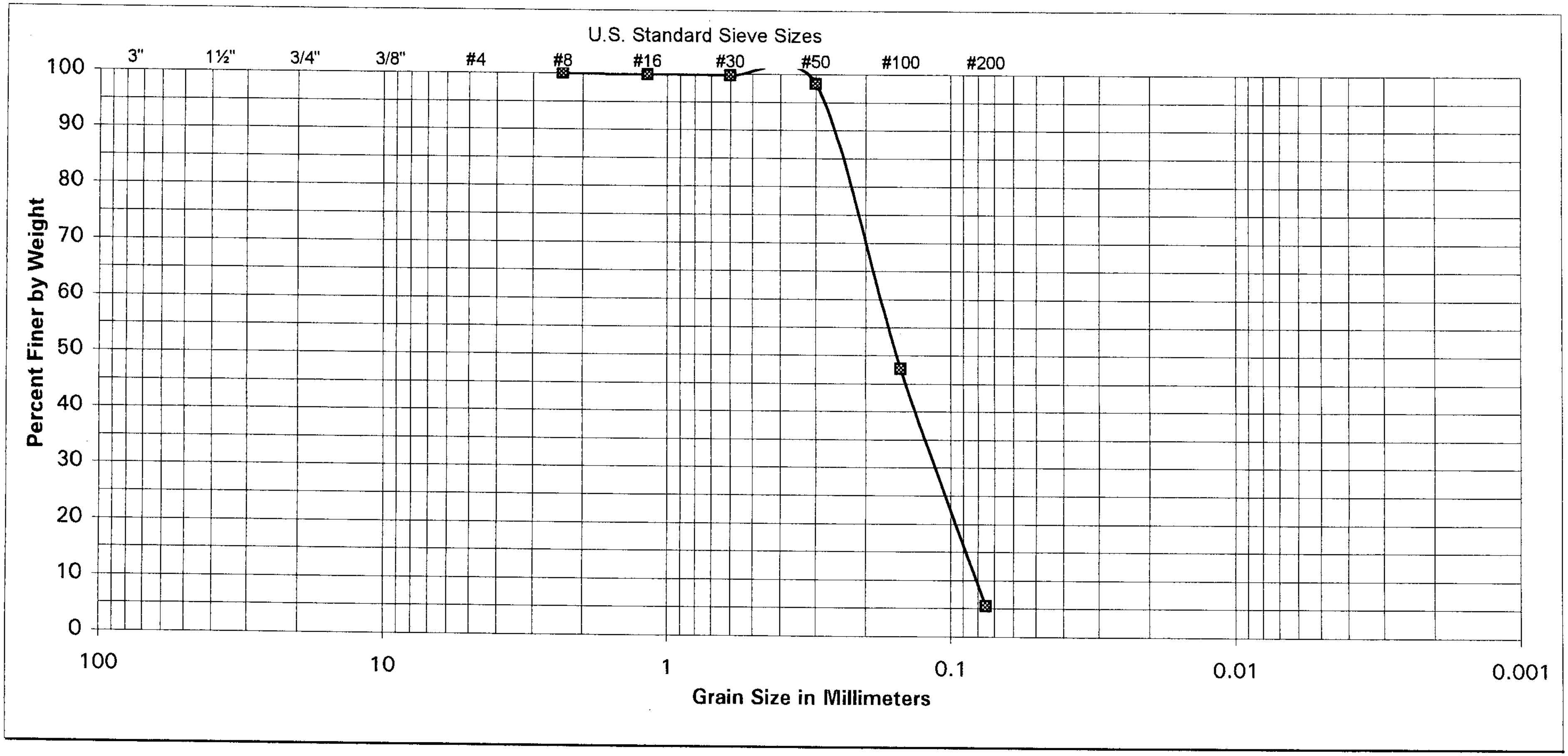
DESCRIPTION: SILT

ATTERBERG LIMITS
LIQUID LIMIT:
PLASTIC LIMIT:
PLASTICITY INDEX:

Geotechnics Incorporated

SOIL CLASSIFICATION

Project No. 0673-002-00
Document No. 02-0022
FIGURE C-1.8



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE	
SAMPLE NUMBER:	B2
SAMPLE LOCATION:	25' - 27'

UNIFIED SOIL CLASSIFICATION: SP-SC

DESCRIPTION: POORLY GRADED SAND WITH CLAY

ATTERBERG LIMITS	
LIQUID LIMIT:	
PLASTIC LIMIT:	
PLASTICITY INDEX:	

Geotechnics Incorporated

SOIL CLASSIFICATION

Project No. 0673-002-00
Document No. 02-0022
FIGURE C-1.9

CHEMICAL TEST RESULTS

SAMPLE	pH	RESISTIVITY (OHM-CM)	SULFATE CONTENT [%]	CHLORIDE CONTENT [%]
B1 @ 5' - 7'	8.0	280	0.08	0.12
B2 @ 2' - 4'	8.1	440	0.18	0.02

UBC TABLE NO. 19-A-4, REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE

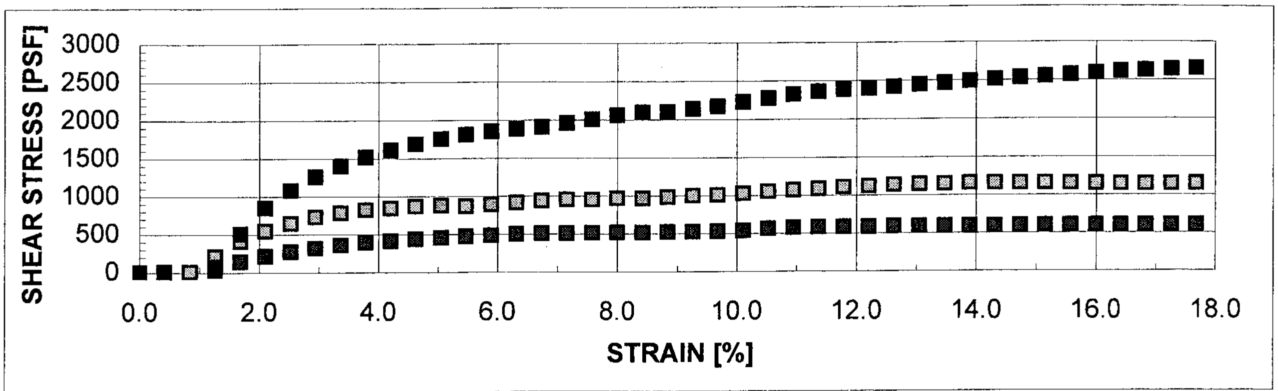
SULFATE CONTENT [%]	SULFATE EXPOSURE	CEMENT TYPE
0.00-0.10	Negligible	-
0.10-0.20	Moderate	II, IP(MS), IS(MS)
0.20-2.00	Severe	V
Above 2.00	Very Severe	V plus pozzolan

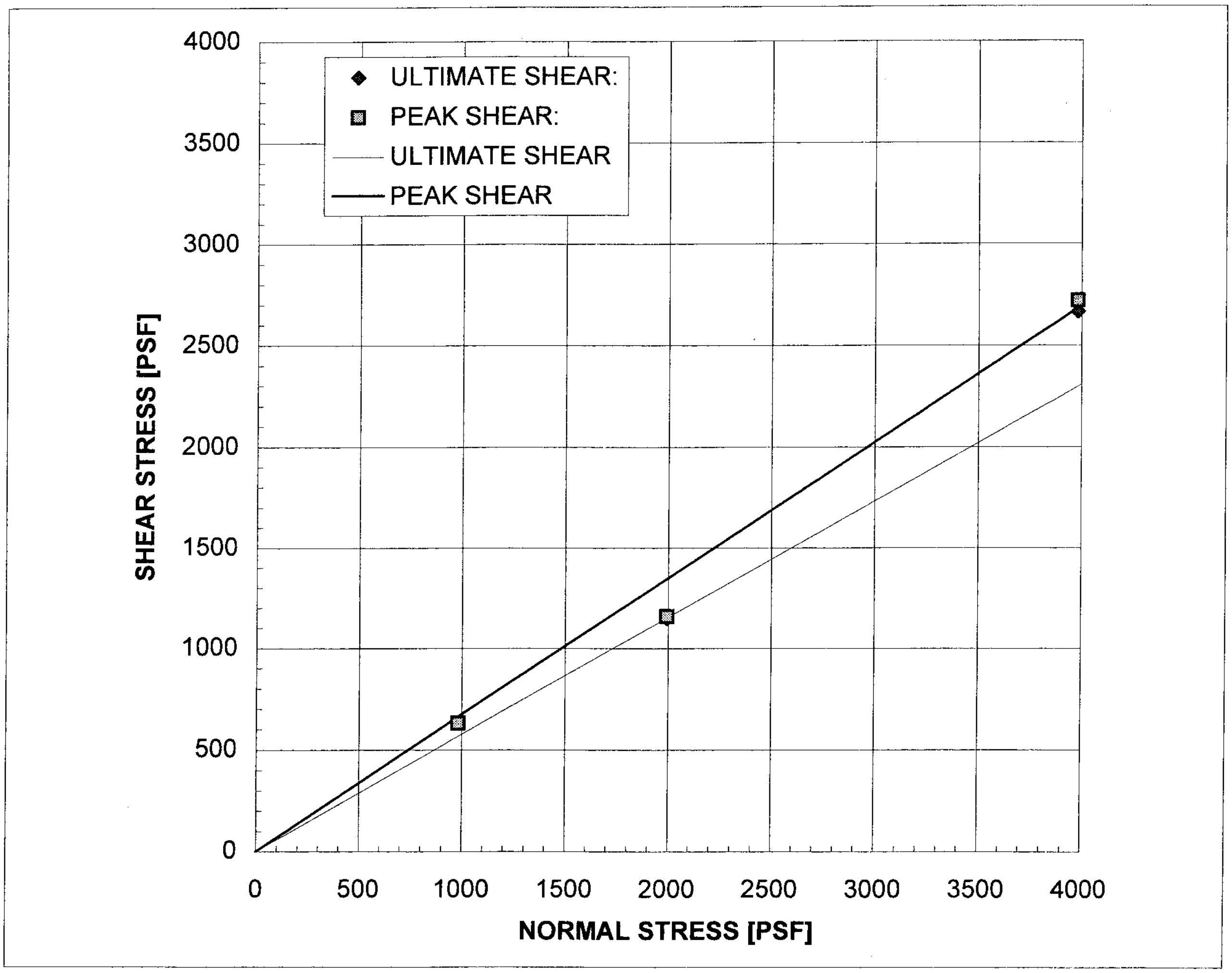
EXPANSION TEST RESULTS
(ASTM D4829)

SAMPLE	DESCRIPTION	EXPANSION INDEX
B2 @ 2' - 4'	Brown lean clay (CL).	69

UBC TABLE NO. 18-1-B, CLASSIFICATION OF EXPANSIVE SOIL

EXPANSION INDEX	POTENTIAL EXPANSION
0-20	Very low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very high





SAMPLE: B1 @ 5' - 7'

LACUSTRINE DEPOSITS (QI):
Brown silt (ML).

	PEAK	ULTIMATE
φ'	34 °	30 °
C'	0 PSF	0 PSF

	IN-SITU	AS-TESTED
γ_d	94.9 PCF	94.9 PCF
w_c	26.3 %	29.2 %

STRAIN RATE: 0.0100 IN/MIN
(Sample was consolidated and drained)

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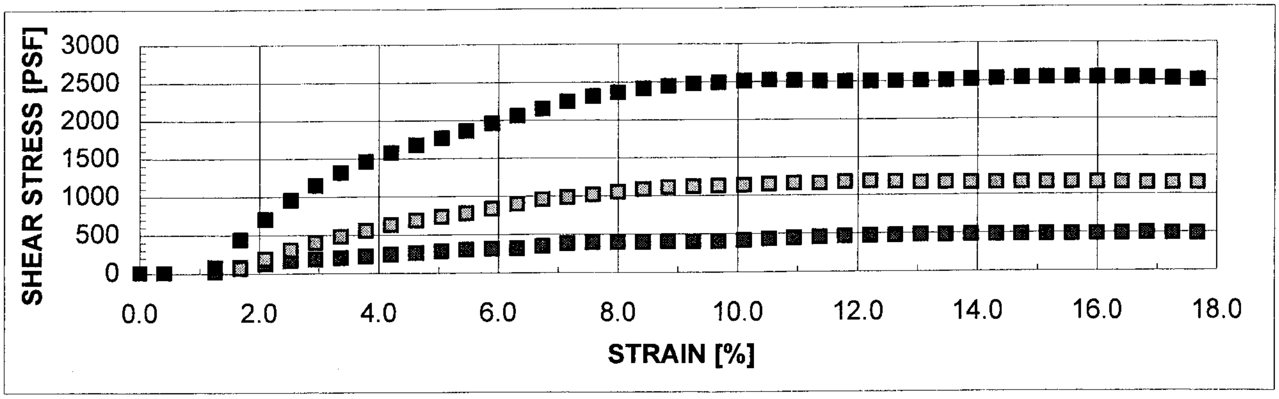
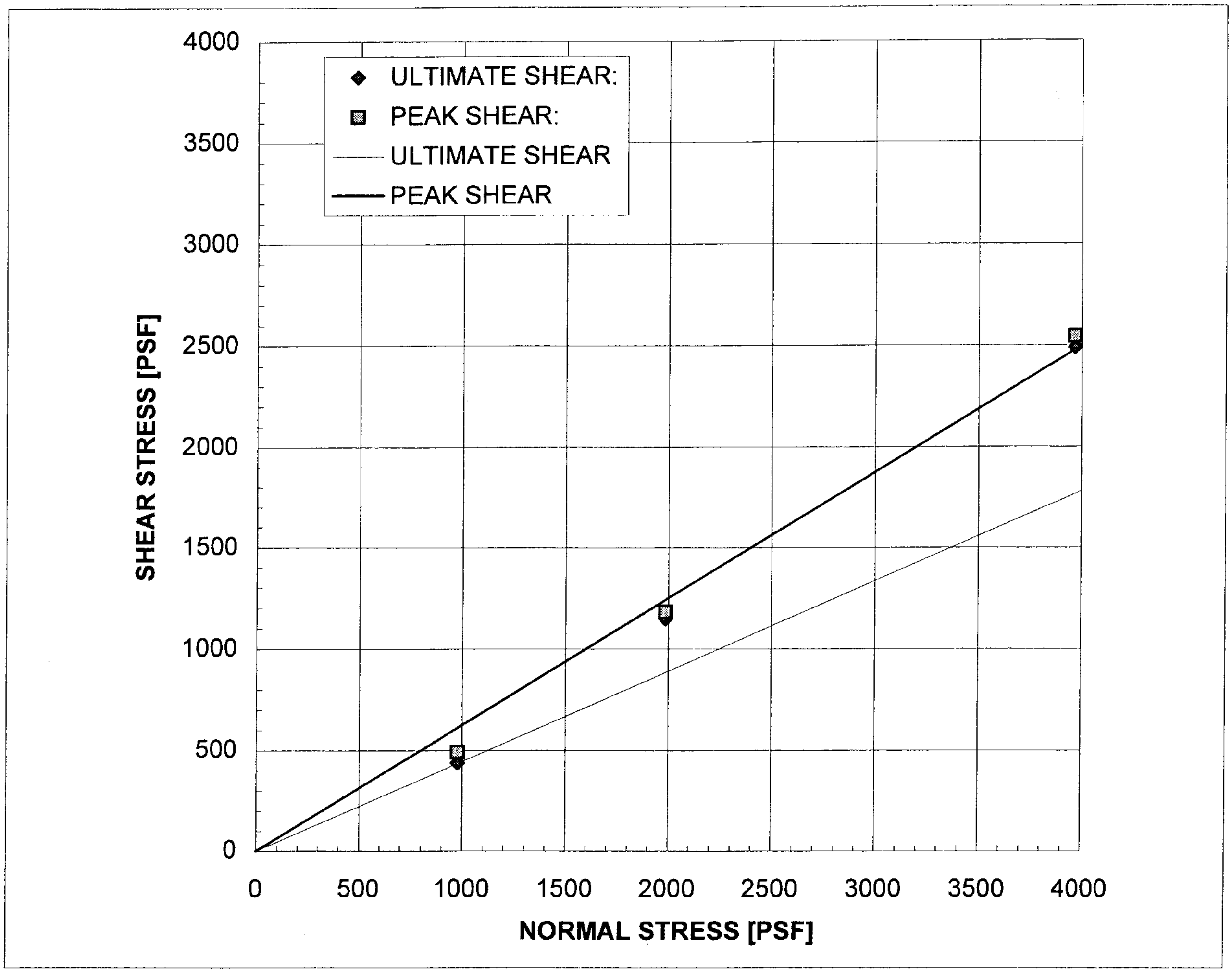
DIRECT SHEAR TEST RESULTS

Project No. 0673-002-00
Document No. 02-0022
FIGURE C-4.1





SAMPLE: B2 @ 10' - 12'

LACUSTRINE DEPOSITS (Ql):
Brown lean clay (CL).

	PEAK	ULTIMATE
ϕ'	32 °	24 °
C'	0 PSF	0 PSF

STRAIN RATE: 0.0100 IN/MIN
(Sample was consolidated and drained)

	IN-SITU	AS-TESTED
γ_d	94.7 PCF	94.7 PCF
w_c	29.5 %	28.4 %

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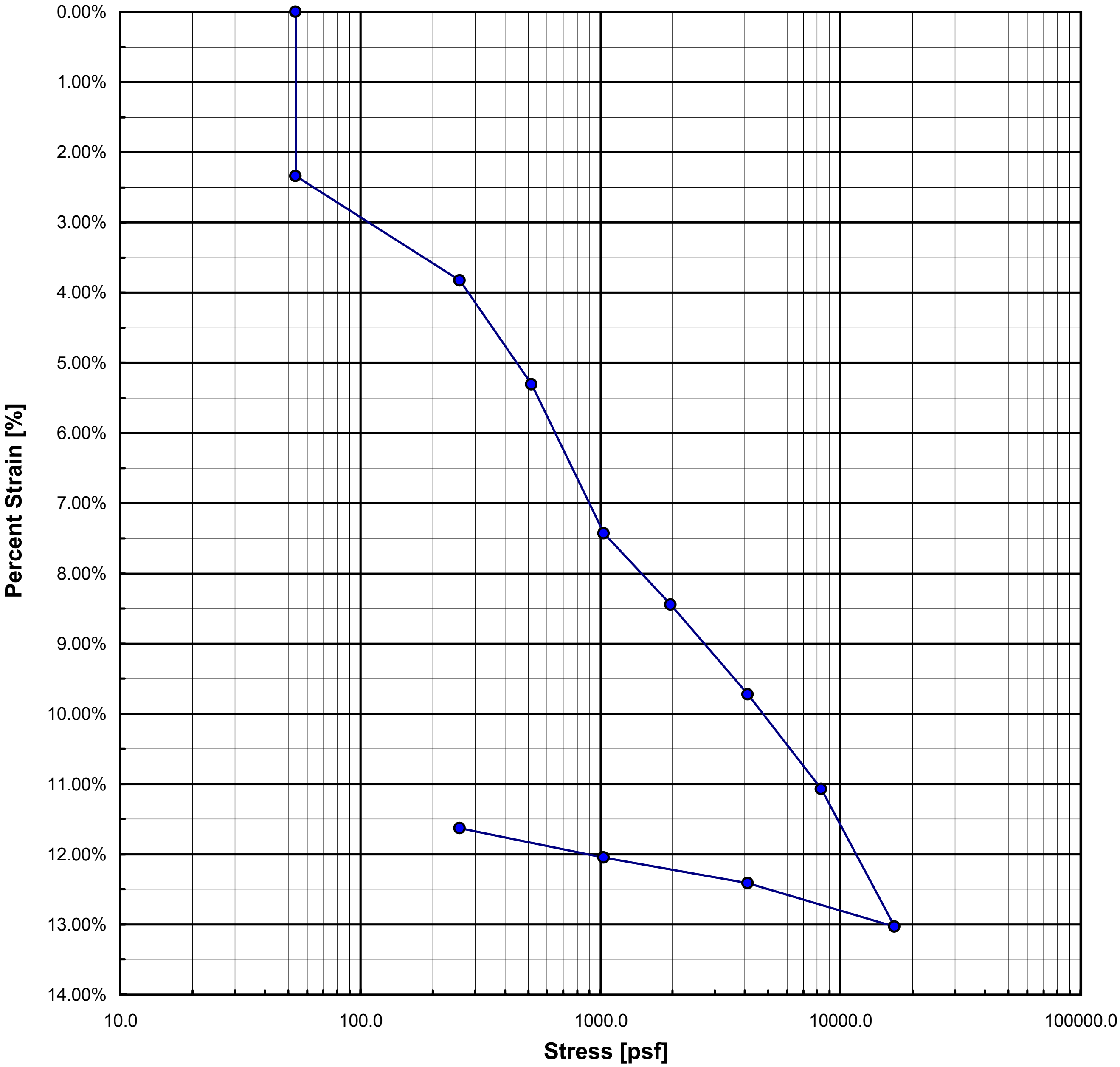
DIRECT SHEAR TEST RESULTS

Project No. 0673-002-00
Document No. 02-0022
FIGURE C-4.2



B2 @ 10' - 12'

INITIAL	FINAL	
1.0000	0.8837	SAMPLE HEIGHT [IN]
95.3	107.8	DRY DENSITY [PCF]
2.77	2.77	SPECIFIC GRAVITY
0.81	0.60	VOID RATIO
29.2	21.8	WATER CONTENT [%]
99.4	100.0	DEGREE OF SATURATION [%]

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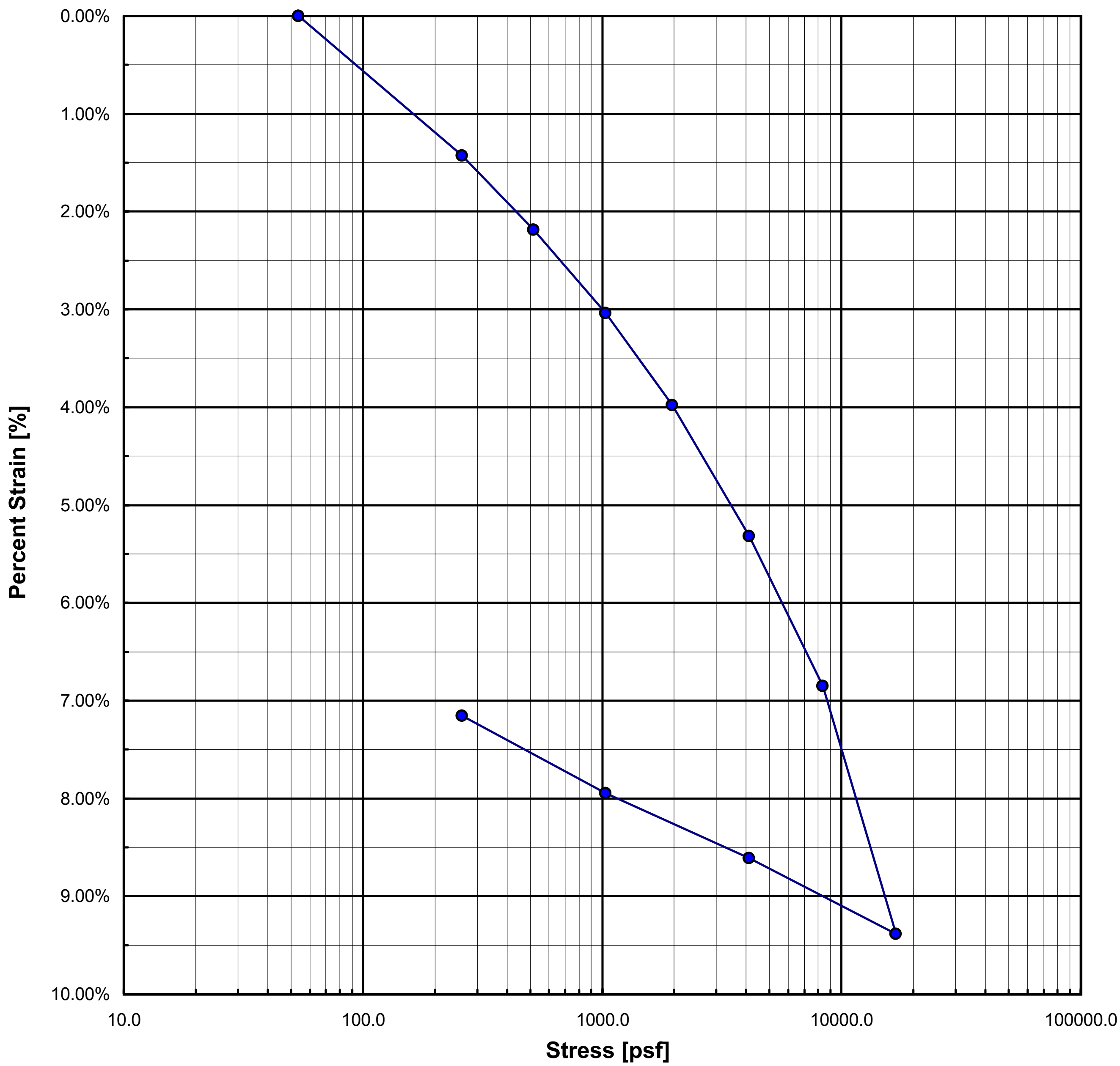
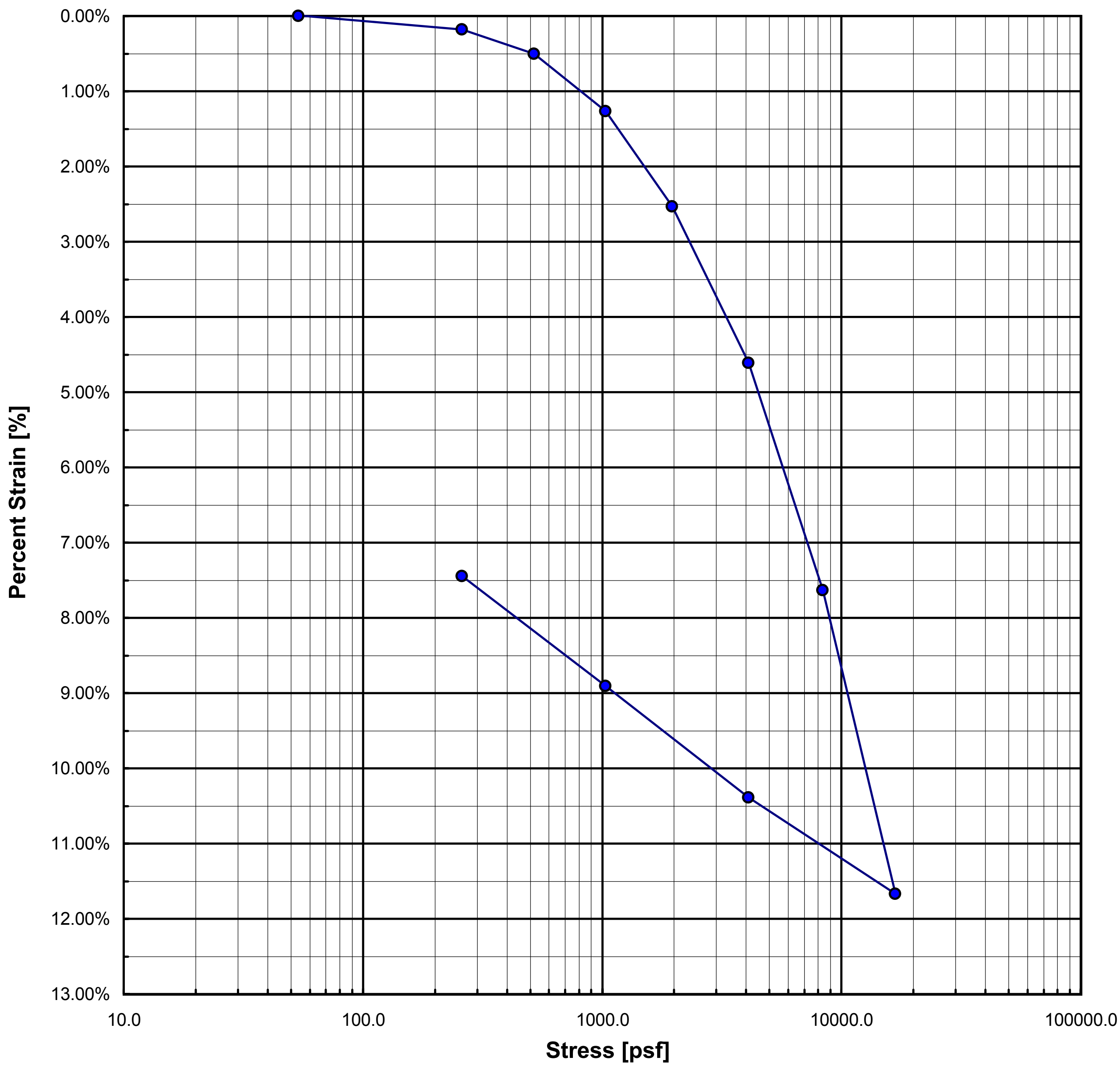
CONSOLIDATION RESULTS

Project No. 0673-002-00
Document No. 02-0022
FIGURE C-5.1



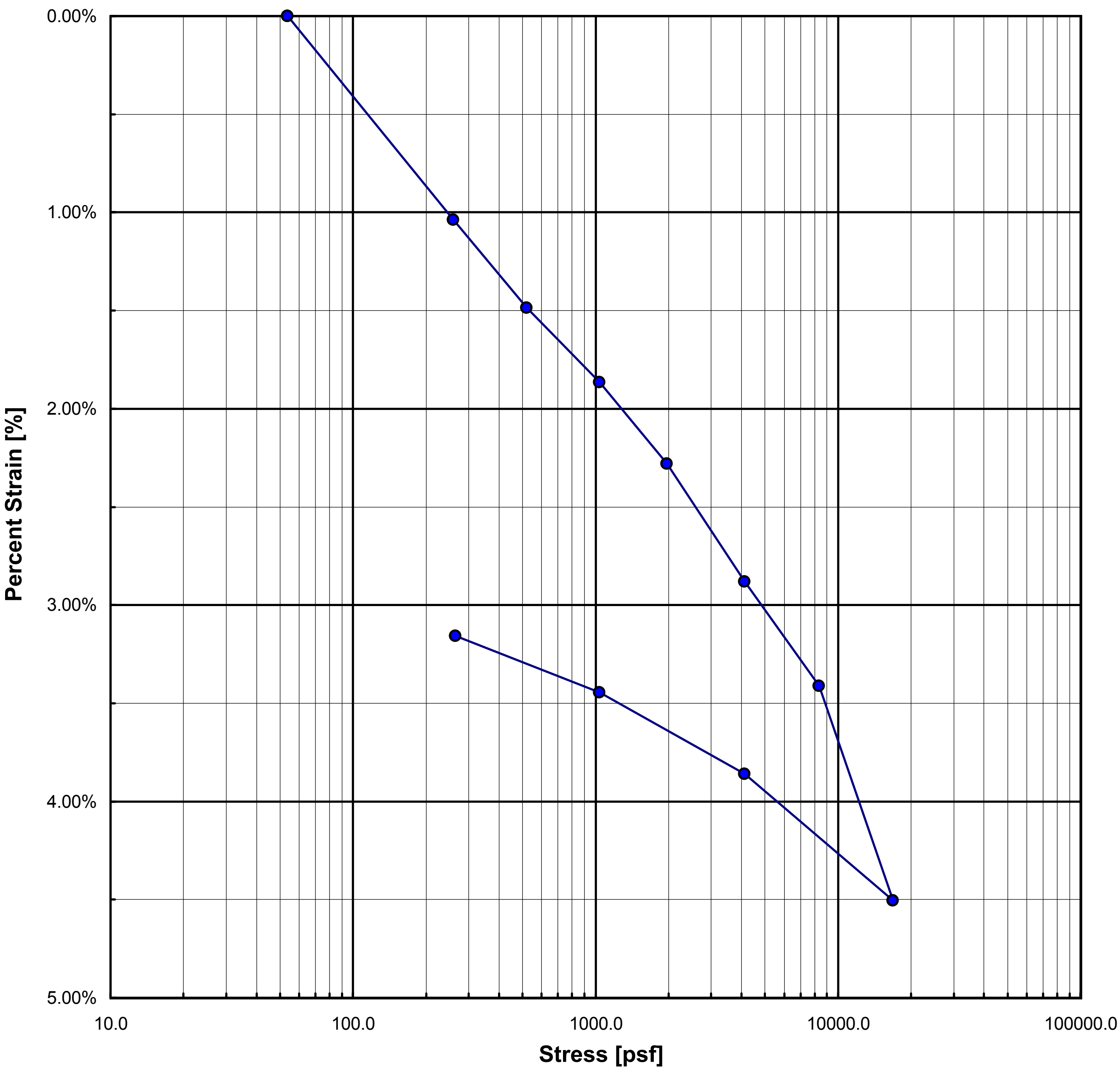
B2 @ 20' - 22'

INITIAL	FINAL	
1.0000	0.9284	SAMPLE HEIGHT [IN]
92.4	99.5	DRY DENSITY [PCF]
2.76	2.76	SPECIFIC GRAVITY
0.87	0.73	VOID RATIO
30.8	26.5	WATER CONTENT [%]
98.2	99.9	DEGREE OF SATURATION [%]



B2 @ 15' - 17'

INITIAL	FINAL	
1.0000	0.9255	SAMPLE HEIGHT [IN]
94.6	102.2	DRY DENSITY [PCF]
2.80	2.80	SPECIFIC GRAVITY
0.85	0.71	VOID RATIO
29.3	25.4	WATER CONTENT [%]
96.8	100.1	DEGREE OF SATURATION [%]



B2 @ 25' - 27'

INITIAL	FINAL	
1.0000	0.9684	SAMPLE HEIGHT [IN]
92.0	95.1	DRY DENSITY [PCF]
2.65	2.65	SPECIFIC GRAVITY
0.80	0.74	VOID RATIO
23.4	25.2	WATER CONTENT [%]
77.7	90.3	DEGREE OF SATURATION [%]

SAMPLE NO.: B2

SAMPLE LOCATION: 2' - 4'

SAMPLE DESCRIPTION: Brown lean clay (CL)

SAMPLE DATE: 11/29/01

TEST DATE: 12/20/01

LABORATORY TEST DATA

	TEST SPECIMEN	1	2	3	4	5	
A	COMPACTOR PRESSURE	60	70	120			[PSI]
B	INITIAL MOISTURE	11.7	11.7	11.7			[%]
C	BATCH SOIL WEIGHT	950	970	980			[G]
D	WATER ADDED	70	60	52			[ML]
E	WATER ADDED (D*(100+B)/C)	8.2	6.9	5.9			[%]
F	COMPACTION MOISTURE (B+E)	19.9	18.6	17.6			[%]
G	MOLD WEIGHT	2113.9	2113.0	2114.6			[G]
H	TOTAL BRIQUETTE WEIGHT	3125.4	3137.4	3139.5			[G]
I	NET BRIQUETTE WEIGHT (H-G)	1011.5	1024.4	1024.9			[G]
J	BRIQUETTE HEIGHT	2.44	2.44	2.43			[IN]
K	DRY DENSITY (30.3*I/((100+F)*J))	104.7	107.3	108.6			[PCF]
L	EXUDATION LOAD	1835	3883	4778			[LB]
M	EXUDATION PRESSURE (L/12.54)	146	310	381			[PSI]
N	STABILOMETER AT 1000 LBS	60	53	42			[PSI]
O	STABILOMETER AT 2000 LBS	134	115	91			[PSI]
P	DISPLACEMENT FOR 100 PSI	6.21	5.49	5.44			[Turns]
Q	R VALUE BY STABILOMETER	7	15	26			
R	CORRECTED R-VALUE (See Fig. 14)	7	14	25			
S	EXPANSION DIAL READING	0.0026	0.0032	0.0046			[IN]
T	EXPANSION PRESSURE (S*43,300)	113	139	199			[PSF]
U	COVER BY STABILOMETER	0.90	0.83	0.72			[FT]
V	COVER BY EXPANSION	0.87	1.07	1.53			[FT]

TRAFFIC INDEX:	4.5
GRAVEL FACTOR:	1.49
UNIT WEIGHT OF COVER [PCF]:	130
R-VALUE BY EXUDATION:	14
R-VALUE BY EXPANSION:	7
R-VALUE AT EQUILIBRIUM:	7

*Note: Gravel factor estimated from pavement section in general accordance with CTM 301, Section C, Part b.

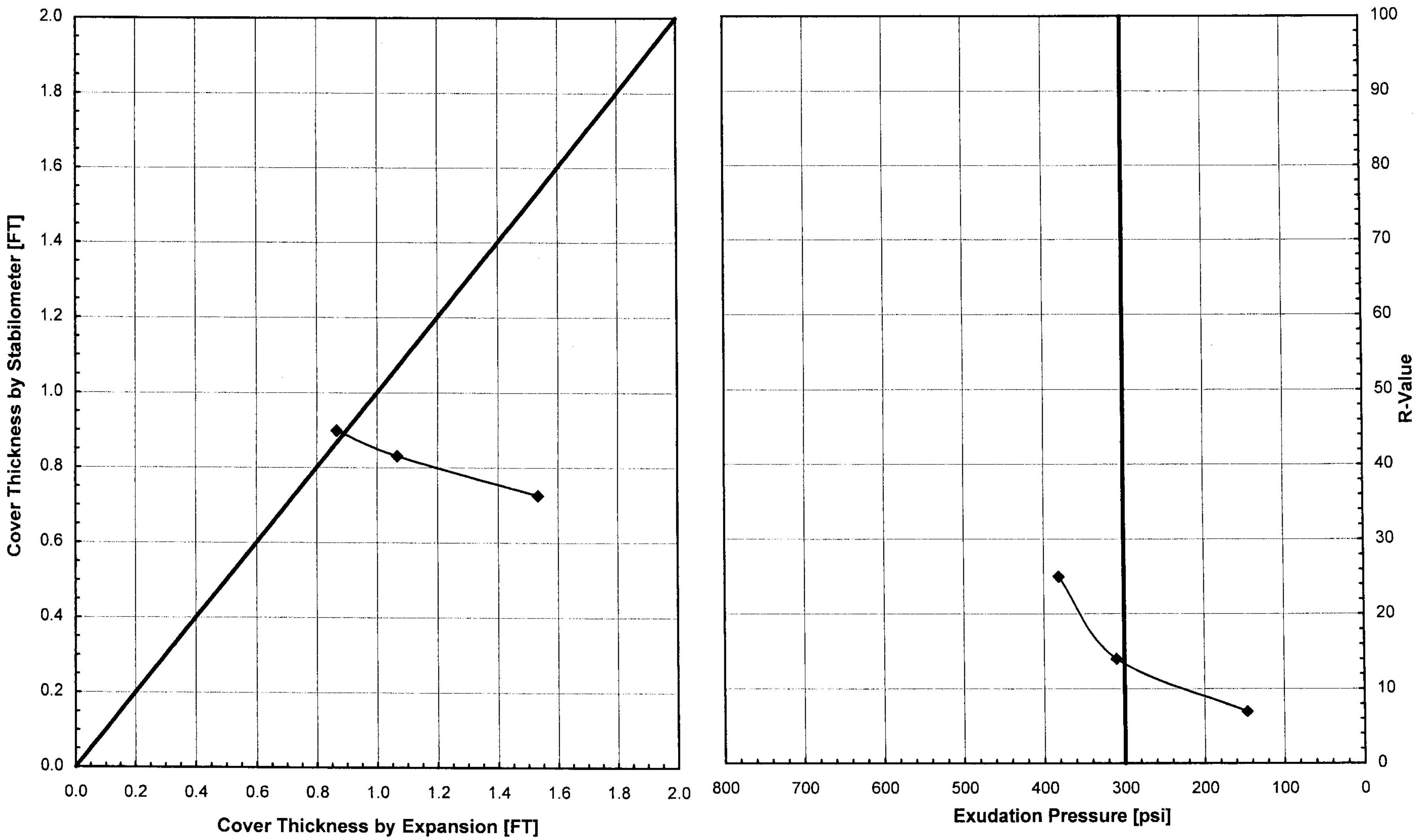
Geotechnics Incorporated

R-VALUE TEST RESULTS

Project No. 0673-002-00
Document No. 02-0022
FIGURE C-6.1

Sample: B2, 2' - 4'

R-Value at Equilibrium: 7



Geotechnics Incorporated

COVER AND EXUDATION CHARTS

Project No. 0673-002-00
Document No. 02-0022
FIGURE C-6.2

PAVEMENT CALCULATION SHEET

(Based On CalTrans Topic 608.4)

Entered Values:

Traffic Index:	4.5
R-Value (S.G.):	7
R-Value (A.B.):	78
R-Value (A.S.):	50
Safety Factor:	0.2
Gf (A.C.):	2.54 [ft]
Gf (A.B.):	1.1 [ft]
Gf (A.S.):	1.0 [ft]

WITH SUBBASE Calculations:

GE (Total):	1.34 [ft]	
GE (A.C.):	0.32 [ft]	
GE (A.C.) + S.F.:	0.52 [ft]	
T (A.C.):	0.20 [ft]	
T (A.C.): (Rounded 0.0	0.20 [ft] =	2.4 [in]
GE (A.C.): (Actual)	0.64 [ft]	
GE (A.C. + A.B.):	0.72 [ft]	
GE (A.C. + A.B.) + S.F.	0.92 [ft]	
GE (A.B.):	0.29 [ft]	
T (A.B.):	0.28 [ft]	
T (A.C.): (Rounded 0.0	0.25 [ft] =	3.0 [in]
GE (A.B.): (Actual)	0.28 [ft]	
GE (A.S.):	0.43 [ft]	
T (A.S.):	0.43 [ft] =	5.2 [in]
GE (A.S.) (Actual):	0.33	
GE (Act. Tot):	1.24	

Ave Gf
1.49

RECOMMENDED PAVEMENT SECTION (WITH SUBBASE)

Use
3
inches asphalt concrete
over
3
inches aggregate base
over
4
inches aggregate subbase

WITHOUT SUBBASE Calculations:

GE (Total):	1.34 [ft]	
GE (A.C.):	0.32 [ft]	
GE (A.C.) + S.F.:	0.52 [ft]	
T (A.C.):	0.20 [ft]	
T (A.C.): (Rounded 0.0	0.20 [ft]	2.4 [in]
GE (A.C.): (Actual)	0.64 [ft]	
GE (A.B.):	0.71 [ft]	
T (A.B.):	0.64 [ft] =	7.7 [in]
GE (A.B.): (Actual)	0.73 [ft]	
GE (Act. Tot):	1.37	

Ave Gf
1.49

RECOMMENDED PAVEMENT SECTION (WITHOUT SUBBASE)

Use
3
inches asphalt concrete
over
8
inches of aggregate base

FULL DEPTH A.C. SECTION Calculations:

GE (Total):	1.34 [ft]	
Gf (A.C.):	2.54 [ft]	
T. (A.C.):	0.53 [ft] =	6.3 [in]
GE (Act. Tot):	1.27	

Ave Gf
2.54

RECOMMENDED FULL DEPTH ASPHALT PAVEMENT SECTION

Use
6
inches A.C. over native

PAVEMENT CALCULATION SHEET

(Based On CalTrans Topic 608.4)

Entered Values:

Traffic Index:	5.0
R-Value (S.G.):	7
R-Value (A.B.):	78
R-Value (A.S.):	50
Safety Factor:	0.2
Gf (A.C.):	2.54 [ft]
Gf (A.B.):	1.1 [ft]
Gf (A.S.):	1.0 [ft]

WITH SUBBASE Calculations:

GE (Total):	1.49 [ft]	
GE (A.C.):	0.35 [ft]	
GE (A.C.) + S.F.:	0.55 [ft]	
T (A.C.):	0.22 [ft]	
T (A.C.): (Rounded 0.0	0.20 [ft] =	2.4 [in]
GE (A.C.): (Actual)	0.64 [ft]	
GE (A.C. + A.B.):	0.80 [ft]	
GE (A.C. + A.B.) + S.F.:	1.00 [ft]	
GE (A.B.):	0.37 [ft]	
T (A.B.):	0.33 [ft]	
T (A.C.): (Rounded 0.0	0.35 [ft] =	4.2 [in]
GE (A.B.): (Actual)	0.28 [ft]	
GE (A.S.):	0.58 [ft]	
T (A.S.):	0.58 [ft] =	6.9 [in]
GE (A.S.) (Actual):	0.33	
GE (Act. Tot):	1.24	

Ave Gf
1.49

RECOMMENDED PAVEMENT SECTION (WITH SUBBASE)

Use
3
inches asphalt concrete
over
3
inches aggregate base
over
4
inches aggregate subbase

WITHOUT SUBBASE Calculations:

GE (Total):	1.49 [ft]	
GE (A.C.):	0.35 [ft]	
GE (A.C.) + S.F.:	0.55 [ft]	
T (A.C.):	0.22 [ft]	
T (A.C.): (Rounded 0.0	0.20 [ft]	2.4 [in]
GE (A.C.): (Actual)	0.64 [ft]	
GE (A.B.):	0.85 [ft]	
T (A.B.):	0.78 [ft] =	9.3 [in]
GE (A.B.): (Actual)	0.92 [ft]	
GE (Act. Tot):	1.55	

Ave Gf
1.43

RECOMMENDED PAVEMENT SECTION (WITHOUT SUBBASE)

Use
3
inches asphalt concrete
over
10
inches of aggregate base

FULL DEPTH A.C. SECTION Calculations:

GE (Total):	1.49 [ft]	
Gf (A.C.):	2.54 [ft]	
T. (A.C.):	0.59 [ft] =	7.0 [in]
GE (Act. Tot):	1.27	

Ave Gf
2.54

RECOMMENDED FULL DEPTH ASPHALT PAVEMENT SECTION

Use
8
inches A.C. over native

Geotechnics Incorporated

PAVEMENT SECTION DESIGN

Project No. 0673-002-00
Document No. 02-0022
FIGURE C-6.4

PAVEMENT CALCULATION SHEET

(Based On CalTrans Topic 608.4)

Entered Values:

Traffic Index:	6.0
R-Value (S.G.):	7
R-Value (A.B.):	78
R-Value (A.S.):	50
Safety Factor:	0.2
Gf (A.C.):	2.32 [ft]
Gf (A.B.):	1.1 [ft]
Gf (A.S.):	1.0 [ft]

WITH SUBBASE Calculations:

GE (Total):	1.79 [ft]	
GE (A.C.):	0.42 [ft]	
GE (A.C.) + S.F.:	0.62 [ft]	
T (A.C.):	0.27 [ft]	
T (A.C.): (Rounded 0.0	0.25 [ft] =	3.0 [in]
GE (A.C.): (Actual)	0.58 [ft]	
GE (A.C. + A.B.):	0.96 [ft]	
GE (A.C. + A.B.) + S.F.	1.16 [ft]	
GE (A.B.):	0.58 [ft]	
T (A.B.):	0.53 [ft]	
T (A.C.): (Rounded 0.0	0.55 [ft] =	6.6 [in]
GE (A.B.): (Actual)	0.28 [ft]	
GE (A.S.):	0.93 [ft]	
T (A.S.):	0.93 [ft] =	11.2 [in]
GE (A.S.) (Actual):	0.33	
GE (Act. Tot):	1.19	

Ave Gf
1.43

RECOMMENDED PAVEMENT SECTION (WITH SUBBASE)

Use
3
inches asphalt concrete
over
3
inches aggregate base
over
4
inches aggregate subbase

WITHOUT SUBBASE Calculations:

GE (Total):	1.79 [ft]	
GE (A.C.):	0.42 [ft]	
GE (A.C.) + S.F.:	0.62 [ft]	
T (A.C.):	0.27 [ft]	
T (A.C.): (Rounded 0.0	0.25 [ft]	3.0 [in]
GE (A.C.): (Actual)	0.77 [ft]	
GE (A.B.):	1.01 [ft]	
T (A.B.):	0.92 [ft] =	11.0 [in]
GE (A.B.): (Actual)	1.01 [ft]	
GE (Act. Tot):	1.78	

Ave Gf
1.43

RECOMMENDED PAVEMENT SECTION (WITHOUT SUBBASE)

Use
4
inches asphalt concrete
over
11
inches of aggregate base

FULL DEPTH A.C. SECTION Calculations:

GE (Total):	1.79 [ft]	
Gf (A.C.):	2.32 [ft]	
T. (A.C.):	0.77 [ft] =	9.2 [in]
GE (Act. Tot):	1.18	

Ave Gf
2.32

RECOMMENDED FULL DEPTH ASPHALT PAVEMENT SECTION

Use
6
inches A.C. over native

Entered Values:

Traffic Index:	7.0
R-Value (S.G.):	7
R-Value (A.B.):	78
R-Value (A.S.):	50
Safety Factor:	0.2
Gf (A.C.):	2.14 [ft]
Gf (A.B.):	1.1 [ft]
Gf (A.S.):	1.0 [ft]

PAVEMENT CALCULATION SHEET

(Based On CalTrans Topic 608.4)

WITH SUBBASE Calculations:

GE (Total):	2.08 [ft]	
GE (A.C.):	0.49 [ft]	
GE (A.C.) + S.F.:	0.69 [ft]	
T (A.C.):	0.32 [ft]	
T (A.C.): (Rounded 0.0	0.30 [ft] =	3.6 [in]
GE (A.C.): (Actual)	0.54 [ft]	
GE (A.C. + A.B.):	1.12 [ft]	
GE (A.C. + A.B.) + S.F.	1.32 [ft]	
GE (A.B.):	0.79 [ft]	
T (A.B.):	0.71 [ft]	
T (A.C.): (Rounded 0.0	0.70 [ft] =	8.4 [in]
GE (A.B.): (Actual)	0.28 [ft]	
GE (A.S.):	1.27 [ft]	
T (A.S.):	1.27 [ft] =	15.3 [in]
GE (A.S.) (Actual):	0.33	
GE (Act. Tot):	1.14	

Ave Gf
1.37

RECOMMENDED PAVEMENT SECTION (WITH SUBBASE)

Use
3
inches asphalt concrete
over
3
inches aggregate base
over
4
inches aggregate subbase

WITHOUT SUBBASE Calculations:

GE (Total):	2.08 [ft]	
GE (A.C.):	0.49 [ft]	
GE (A.C.) + S.F.:	0.69 [ft]	
T (A.C.):	0.32 [ft]	
T (A.C.): (Rounded 0.0	0.30 [ft]	3.6 [in]
GE (A.C.): (Actual)	0.71 [ft]	
GE (A.B.):	1.37 [ft]	
T (A.B.):	1.25 [ft] =	14.9 [in]
GE (A.B.): (Actual)	1.38 [ft]	
GE (Act. Tot):	2.09	

Ave Gf
1.32

RECOMMENDED PAVEMENT SECTION (WITHOUT SUBBASE)

Use
4
inches asphalt concrete
over
15
inches of aggregate base

FULL DEPTH A.C. SECTION Calculations:

GE (Total):	2.08 [ft]	
Gf (A.C.):	2.14 [ft]	
T. (A.C.):	0.97 [ft] =	11.7 [in]
GE (Act. Tot):	1.07	

Ave Gf
2.14

RECOMMENDED FULL DEPTH ASPHALT PAVEMENT SECTION

Use
6
inches A.C. over native

Geotechnics Incorporated

PAVEMENT SECTION DESIGN

Project No. 0673-002-00
Document No. 02-0022
FIGURE C-6.6

APPENDIX D

SEISMIC ANALYSIS

Seismic analysis was conducted for the subject site in order to develop parameters for structural design and liquefaction analysis. This appendix presents the raw data from our analysis from three commercially available computer programs, EQFAULT, EQSEARCH, and FRISKSP (Blake, 1998). All three analyses used the same published attenuation relationship for deep soil sites (Sadigh, 1997).

EQSEARCH: The program EQSEARCH was used to generate a table of estimated characteristics of nearby seismic events which were recorded between 1800 and 1995. This table is presented in Appendix D, and shows the epicenters, magnitudes, and dates of these nearby earthquakes, along with the estimated peak ground acceleration for the site.

EQFAULT: The program EQFAULT was used to develop the deterministic peak ground acceleration parameters summarized in Table 1 of the formal report.

FRISKSP: The program FRISKSP was used perform a probabilistic analysis of seismicity at the subject site based on the characteristic earthquake distribution of Youngs and Coppersmith (1985). The results are also presented in Appendix D. The probabilistic analysis was used to define the Upper Bound and Design Basis Earthquakes at the site for use in structural design. The first set of graphs do not incorporate Magnitude Weighting Factors, and represent the probabilistic values presented in the text of this report. The second set of graphs do incorporate magnitude weighting factors for use in the liquefaction analysis (which is based on a magnitude 7.5 normalized event).

DATE: Friday, January 25, 2002

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*************************************
*                                   *
*          E Q S E A R C H          *
*                                   *
*             Ver. 2.20             *
*                                   *
*                                   *
*************************************
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(Estimation of Peak Horizontal Acceleration
From California Earthquake Catalogs)

SEARCH PERFORMED FOR: CalEnergy Operting Company, Inc.

JOB NUMBER: 0673-002-00

JOB NAME: CalEnergy Geothermal Plant Unit No. 6

SITE COORDINATES:
LATITUDE: 33.168 N
LONGITUDE: 115.619 W

TYPE OF SEARCH: RADIUS
SEARCH RADIUS: 62 mi

SEARCH MAGNITUDES: 5.0 TO 9.0

SEARCH DATES: 1800 TO 1995

ATTENUATION RELATION: 18) Idriss (1994) Horiz. - Deep Soil

UNCERTAINTY (M=Mean, S=Mean+1-Sigma): M

SCOND: 0

FAULT TYPE ASSUMED (DS=Reverse, SS=Strike-Slip): SS

COMPUTE PEAK HORIZONTAL ACCELERATION

EARTHQUAKE-DATA FILE USED: ALLQUAKE.DAT

TIME PERIOD OF EXPOSURE FOR STATISTICAL COMPARISON: 25 years

SOURCE OF DEPTH VALUES (A=Attenuation File, E=Earthquake Catalog): A

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (GMT) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	32.500	115.000	11/29/1852	20 0 0.0	10.0	6.50	0.032	V	58 [94]
T-A	33.500	115.820	5/ 0/1868	0 0 0.0	10.0	6.30	0.077	VII	26 [41]
DMG	33.000	115.000	5/ 3/1872	1 0 0.0	10.0	5.50	0.026	V	38 [61]
DMG	33.400	116.300	2/ 9/1890	12 6 0.0	10.0	6.30	0.042	VI	42 [68]
DMG	32.700	116.300	2/24/1892	720 0.0	10.0	6.70	0.045	VI	51 [82]
DMG	33.200	116.200	5/28/1892	1115 0.0	10.0	6.30	0.056	VI	34 [54]
DMG	32.500	115.500	4/19/1906	030 0.0	10.0	6.00	0.028	V	47 [75]
DMG	32.800	115.500	6/23/1915	359 0.0	10.0	6.25	0.072	VII	26 [42]
DMG	32.800	115.500	6/23/1915	456 0.0	10.0	6.25	0.072	VII	26 [42]
DMG	33.500	116.500	9/30/1916	211 0.0	10.0	5.00	0.009	III	56 [90]
DMG	32.800	115.300	5/28/1917	6 6 0.0	10.0	5.50	0.033	V	31 [51]
DMG	32.500	115.500	5/ 1/1918	432 0.0	10.0	5.00	0.012	III	47 [75]
MGI	33.200	116.600	10/12/1920	1748 0.0	10.0	5.30	0.012	III	57 [91]
DMG	32.500	115.500	9/ 8/1921	1924 0.0	10.0	5.00	0.012	III	47 [75]
DMG	32.500	115.500	11/ 5/1923	22 7 0.0	10.0	5.00	0.012	III	47 [75]
DMG	32.500	115.500	11/ 7/1923	2357 0.0	10.0	5.50	0.019	IV	47 [75]
MGI	32.500	115.500	4/16/1925	330 0.0	10.0	5.00	0.012	III	47 [75]
MGI	32.500	115.500	4/16/1925	520 0.0	10.0	5.30	0.016	IV	47 [75]
DMG	34.000	116.000	4/ 3/1926	20 8 0.0	10.0	5.50	0.012	III	61 [99]
DMG	32.500	115.500	1/ 1/1927	81645.0	10.0	5.75	0.023	IV	47 [75]
DMG	32.500	115.500	1/ 1/1927	91330.0	10.0	5.50	0.019	IV	47 [75]
MGI	32.700	115.500	1/ 1/1927	13 0 0.0	10.0	5.30	0.026	V	33 [53]
DMG	34.000	116.000	9/ 5/1928	1442 0.0	10.0	5.00	0.008	II	61 [99]
DMG	32.900	115.700	10/ 2/1928	19 1 0.0	10.0	5.00	0.043	VI	19 [31]
DMG	33.000	115.500	2/26/1930	230 0.0	10.0	5.00	0.062	VI	13 [22]
DMG	32.900	115.217	9/ 8/1935	17 3 0.0	10.0	5.00	0.024	V	30 [48]
DMG	32.900	115.217	10/11/1935	14 6 0.0	10.0	5.00	0.024	V	30 [48]
DMG	33.167	115.500	12/20/1935	745 0.0	10.0	5.00	0.101	VII	7 [11]
DMG	33.408	116.261	3/25/1937	1649 1.8	10.0	6.00	0.034	V	41 [65]
DMG	32.900	115.217	6/ 6/1938	242 0.0	10.0	5.00	0.024	V	30 [48]
DMG	32.733	115.500	5/19/1940	43640.9	10.0	6.70	0.081	VII	31 [50]
DMG	32.767	115.483	5/19/1940	455 0.0	10.0	5.50	0.037	V	29 [46]
DMG	32.767	115.483	5/19/1940	55134.0	10.0	5.50	0.037	V	29 [46]
DMG	32.767	115.483	5/19/1940	63320.0	10.0	5.00	0.025	V	29 [46]
DMG	32.767	115.483	5/19/1940	63540.0	10.0	5.50	0.037	V	29 [46]
DMG	33.000	116.433	6/ 4/1940	1035 8.3	10.0	5.10	0.012	III	48 [78]
DMG	34.000	115.750	3/ 3/1942	1 324.0	10.0	5.00	0.008	III	58 [93]
DMG	32.983	115.983	5/23/1942	154729.0	10.0	5.00	0.031	V	25 [40]
DMG	32.967	116.000	10/21/1942	162213.0	10.0	6.50	0.086	VII	26 [42]
DMG	32.967	116.000	10/21/1942	162519.0	10.0	5.00	0.029	V	26 [42]
DMG	32.967	116.000	10/21/1942	162654.0	10.0	5.00	0.029	V	26 [42]
DMG	33.233	115.717	10/22/1942	15038.0	10.0	5.50	0.132	VIII	7 [12]
DMG	32.967	116.000	10/22/1942	181326.0	10.0	5.00	0.029	V	26 [42]
DMG	33.217	116.133	8/15/1945	175624.0	10.0	5.70	0.041	V	30 [48]
DMG	33.000	115.833	1/ 8/1946	185418.0	10.0	5.40	0.065	VI	17 [27]
DMG	34.017	115.683	5/ 2/1949	112547.0	10.0	5.90	0.018	IV	59 [95]
DMG	33.117	115.567	7/28/1950	175048.0	10.0	5.40	0.147	VIII	5 [7]
DMG	33.117	115.567	7/29/1950	143632.0	10.0	5.50	0.156	VIII	5 [7]
DMG	32.983	115.733	1/24/1951	717 2.6	10.0	5.60	0.087	VII	14 [23]
DMG	32.950	115.717	6/14/1953	41729.9	10.0	5.50	0.073	VII	16 [26]
DMG	33.283	116.183	3/19/1954	95429.0	10.0	6.20	0.052	VI	34 [54]
DMG	33.283	116.183	3/19/1954	95556.0	10.0	5.00	0.020	IV	34 [54]
DMG	33.283	116.183	3/19/1954	102117.0	10.0	5.50	0.030	V	34 [54]
DMG	33.283	116.183	3/23/1954	41450.0	10.0	5.10	0.022	IV	34 [54]
DMG	33.000	115.500	12/17/1955	6 729.0	10.0	5.40	0.081	VII	13 [22]
DMG	33.216	115.808	4/25/1957	215738.7	10.0	5.20	0.082	VII	11 [18]
DMG	33.183	115.850	4/25/1957	222412.0	10.0	5.10	0.067	VI	13 [22]
DMG	33.231	116.004	5/26/1957	155933.6	10.0	5.00	0.035	V	23 [36]

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (GMT) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	33.190	116.129	4/ 9/1968	22859.1	10.0	6.40	0.070	VI	30 [47]
DMG	33.113	116.037	4/ 9/1968	3 353.5	10.0	5.20	0.037	V	24 [39]
DMG	33.343	116.346	4/28/1969	232042.9	10.0	5.80	0.026	V	44 [70]
DMG	33.033	115.821	9/30/1971	224611.3	10.0	5.10	0.060	VI	15 [24]
PAS	32.614	115.318	10/15/1979	231653.4	10.0	6.60	0.053	VI	42 [68]
PAS	32.766	115.441	10/15/1979	231930.0	10.0	5.20	0.029	V	30 [48]
PAS	32.927	115.540	10/16/1979	54910.2	10.0	5.10	0.052	VI	17 [28]
PAS	32.928	115.539	10/16/1979	61948.7	10.0	5.10	0.052	VI	17 [28]
PAS	33.014	115.555	10/16/1979	65842.8	10.0	5.50	0.100	VII	11 [18]
PAS	33.501	116.513	2/25/1980	104738.5	10.0	5.50	0.014	IV	56 [91]
PAS	33.098	115.632	4/26/1981	12 928.4	10.0	5.70	0.170	VIII	5 [8]
PAS	32.393	115.311	2/ 7/1987	34514.8	10.0	5.40	0.013	III	56 [91]
PAS	33.082	115.775	11/24/1987	15414.5	10.0	5.80	0.123	VII	11 [17]
PAS	33.013	115.839	11/24/1987	131556.5	10.0	6.00	0.096	VII	17 [27]
GSP	33.876	116.267	6/29/1992	160142.8	10.0	5.20	0.009	III	61 [99]

****

-END OF SEARCH- 73 RECORDS FOUND

COMPUTER TIME REQUIRED FOR EARTHQUAKE SEARCH: 0.4 minutes

MAXIMUM SITE ACCELERATION DURING TIME PERIOD 1800 TO 1995: 0.170g

MAXIMUM SITE INTENSITY (MM) DURING TIME PERIOD 1800 TO 1995: VIII

MAXIMUM MAGNITUDE ENCOUNTERED IN SEARCH: 6.70

NEAREST HISTORICAL EARTHQUAKE WAS ABOUT 5 MILES AWAY FROM SITE.

NUMBER OF YEARS REPRESENTED BY SEARCH: 196 years

RESULTS OF PROBABILITY ANALYSES

TIME PERIOD OF SEARCH: 1800 TO 1995
LENGTH OF SEARCH TIME: 196 years
ATTENUATION RELATION: 18) Idriss (1994) Horiz. - Deep Soil
*** TIME PERIOD OF EXPOSURE FOR PROBABILITY: 25 years

PROBABILITY OF EXCEEDANCE FOR ACCELERATION

ACC. g	NO.OF TIMES EXCED	AVE. OCCUR. #/yr	RECURR. INTERV. years	COMPUTED PROBABILITY OF EXCEEDANCE in 0.5 yr	in 1 yr	in 10 yr	in 50 yr	in 75 yr	in 100 yr	in *** yr
0.01	69	0.352	2.841	0.1614	0.2967	0.9704	1.0000	1.0000	1.0000	0.9998
0.02	56	0.286	3.500	0.1331	0.2485	0.9426	1.0000	1.0000	1.0000	0.9992
0.03	41	0.209	4.780	0.0993	0.1888	0.8765	1.0000	1.0000	1.0000	0.9946
0.04	31	0.158	6.323	0.0760	0.1463	0.7944	0.9996	1.0000	1.0000	0.9808
0.05	27	0.138	7.259	0.0666	0.1287	0.7478	0.9990	1.0000	1.0000	0.9681
0.06	22	0.112	8.909	0.0546	0.1062	0.6745	0.9963	0.9998	1.0000	0.9396
0.07	17	0.087	11.529	0.0424	0.0831	0.5799	0.9869	0.9985	0.9998	0.8856
0.08	13	0.066	15.077	0.0326	0.0642	0.4848	0.9637	0.9931	0.9987	0.8095
0.09	8	0.041	24.500	0.0202	0.0400	0.3351	0.8701	0.9532	0.9831	0.6396
0.10	6	0.031	32.667	0.0152	0.0301	0.2637	0.7836	0.8993	0.9532	0.5348
0.11	5	0.026	39.200	0.0127	0.0252	0.2252	0.7207	0.8524	0.9220	0.4715
0.12	5	0.026	39.200	0.0127	0.0252	0.2252	0.7207	0.8524	0.9220	0.4715
0.13	4	0.020	49.000	0.0102	0.0202	0.1846	0.6396	0.7836	0.8701	0.3996
0.14	3	0.015	65.333	0.0076	0.0152	0.1419	0.5348	0.6827	0.7836	0.3180
0.15	2	0.010	98.000	0.0051	0.0102	0.0970	0.3996	0.5348	0.6396	0.2252
0.16	1	0.005	196.000	0.0025	0.0051	0.0497	0.2252	0.3180	0.3996	0.1198
0.17	1	0.005	196.000	0.0025	0.0051	0.0497	0.2252	0.3180	0.3996	0.1198

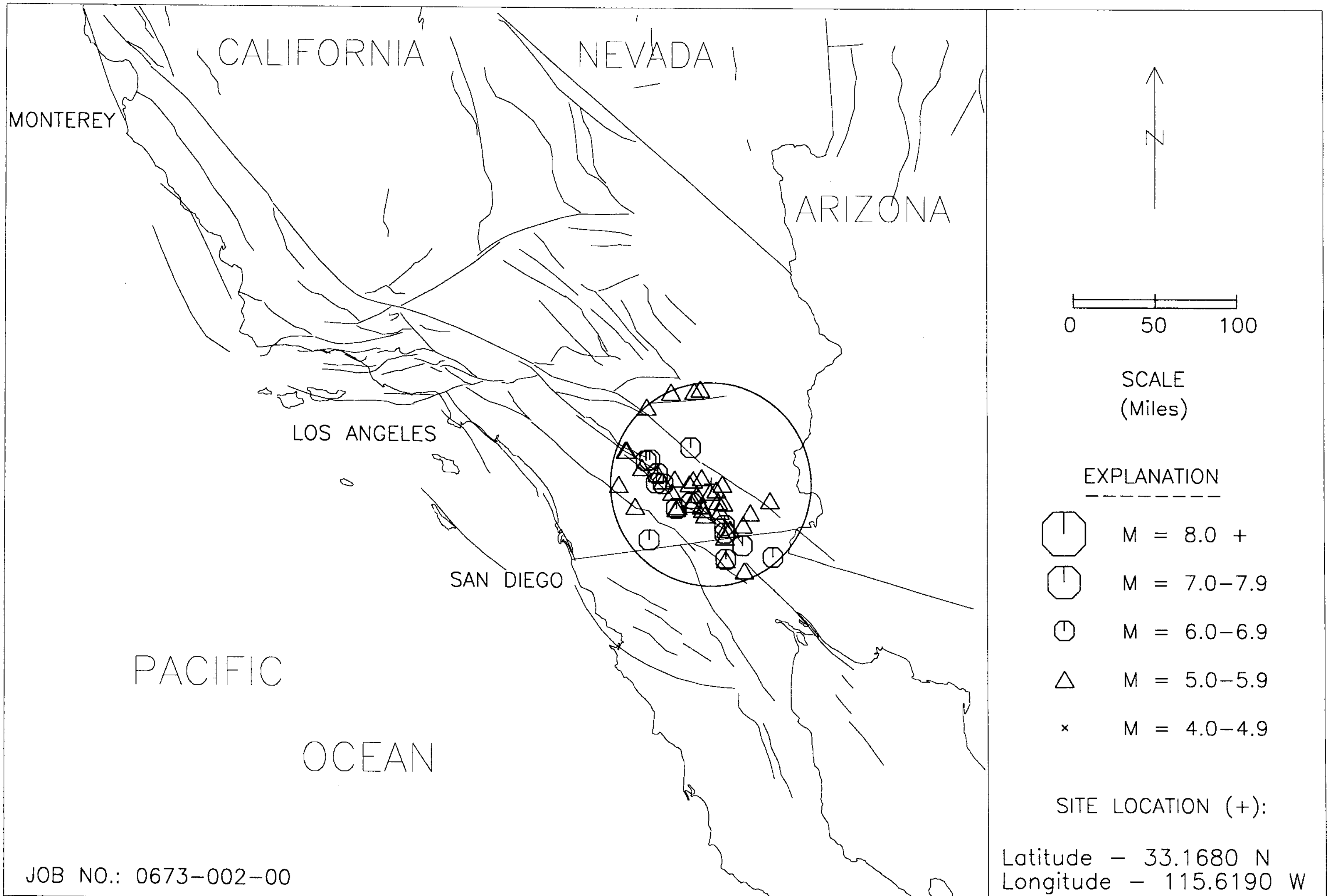
PROBABILITY OF EXCEEDANCE FOR MAGNITUDE

MAG.	NO.OF TIMES EXCED	AVE. OCCUR. #/yr	RECURR. INTERV. years	COMPUTED PROBABILITY OF EXCEEDANCE in 0.5 yr	in 1 yr	in 10 yr	in 50 yr	in 75 yr	in 100 yr	in *** yr
5.00	73	0.372	2.685	0.1699	0.3110	0.9759	1.0000	1.0000	1.0000	0.9999
5.50	36	0.184	5.444	0.0877	0.1678	0.8407	0.9999	1.0000	1.0000	0.9899
6.00	15	0.077	13.067	0.0375	0.0737	0.5348	0.9782	0.9968	0.9995	0.8524
6.50	5	0.026	39.200	0.0127	0.0252	0.2252	0.7207	0.8524	0.9220	0.4715

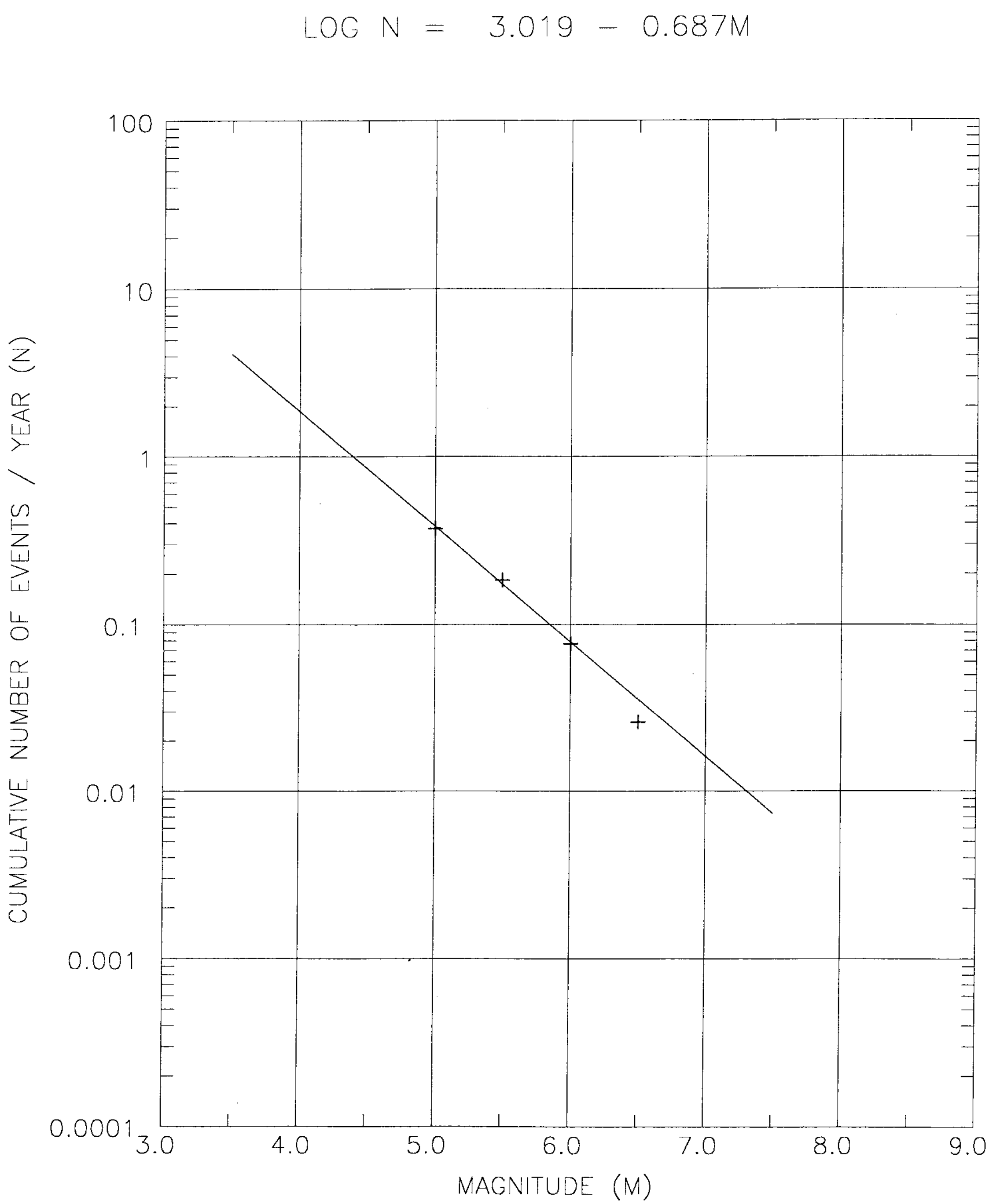
GUTENBERG & RICHTER RECURRENCE RELATIONSHIP:

a-value= 3.019
b-value= 0.687

beta-value= 1.582



HISTORICAL EARTHQUAKES 1800 TO 1995



SEISMIC RECURRENCE CURVE

HISTORICAL EARTHQUAKES FROM 1800 TO 1995

CalEnergy Geothermal Plant Unit No. 6

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***********************
*                     *
*    E Q F A U L T    *
*                     *
*    Version 3.00     *
*                     *
***********************
```

DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 0673-002-00

DATE: 02-04-2002

JOB NAME: CalEnergy Geothermal Unit 6

CALCULATION NAME: Salton Sea Geotehrmal Plant Unit 6

FAULT-DATA-FILE NAME: C:\Program Files\EQFAULT1\cdmgfte2.dat

SITE COORDINATES:
SITE LATITUDE: 33.1680
SITE LONGITUDE: 115.6190

SEARCH RADIUS: 62 mi

ATTENUATION RELATION: 20) Sadigh et al. (1997) Horiz. - Soil
UNCERTAINTY (M=Median, S=Sigma): M Number of Sigmas: 0.0
DISTANCE MEASURE: clodis
SCOND: 0
Basement Depth: 5.00 km Campbell SSR: Campbell SHR:
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: C:\Program Files\EQFAULT1\cdmgfte2.dat

MINIMUM DEPTH VALUE (km): 0.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

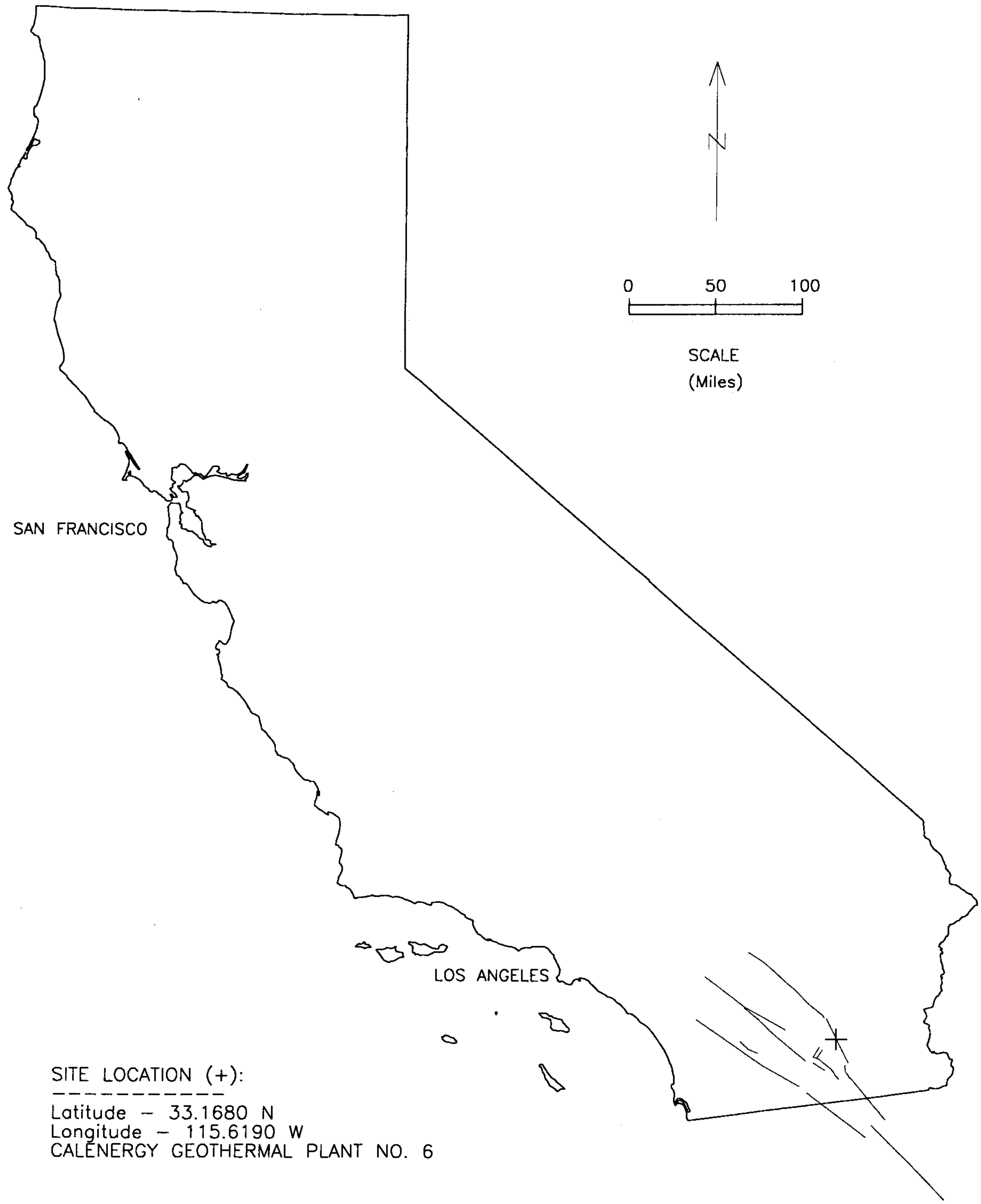
ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
BRAWLEY SEISMIC ZONE	1.2(2.0)	6.4	0.463	X
ELMORE RANCH (EAST)	10.1(16.2)	6.1	0.144	VIII
ELMORE RANCH (WEST)	10.7(17.2)	6.2	0.147	VIII
SAN ANDREAS - Southern	13.7(22.0)	7.4	0.228	IX
SAN ANDREAS - Coachella	14.1(22.7)	7.1	0.195	VIII
SUPERSTITION HILLS (San Jacinto)	15.3(24.7)	6.6	0.139	VIII
IMPERIAL (MODEL A)	16.3(26.3)	6.9	0.155	VIII
IMPERIAL (MODEL B)	17.1(27.6)	6.9	0.148	VIII
SUPERSTITION MTN. (San Jacinto)	18.9(30.4)	6.3	0.090	VII
SAN JACINTO-COYOTE CREEK	21.9(35.3)	7.1	0.131	VIII
SAN JACINTO - BORREGO	23.3(37.5)	6.6	0.089	VII
SAN JACINTO-ANZA	29.8(48.0)	7.2	0.101	VII
ELSINORE-COYOTE MOUNTAIN	35.0(56.3)	6.8	0.063	VI
LAGUNA SALADA	35.7(57.5)	7.1	0.077	VII
ELSINORE-JULIAN	45.2(72.7)	7.1	0.057	VI
EARTHQUAKE VALLEY	46.2(74.4)	6.5	0.035	V
CERRO PRIETO	54.6(87.8)	7.1	0.045	VI

****

-END OF SEARCH- 17 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE BRAWLEY SEISMIC ZONE FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 1.2 MILES (2.0 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.4626 g

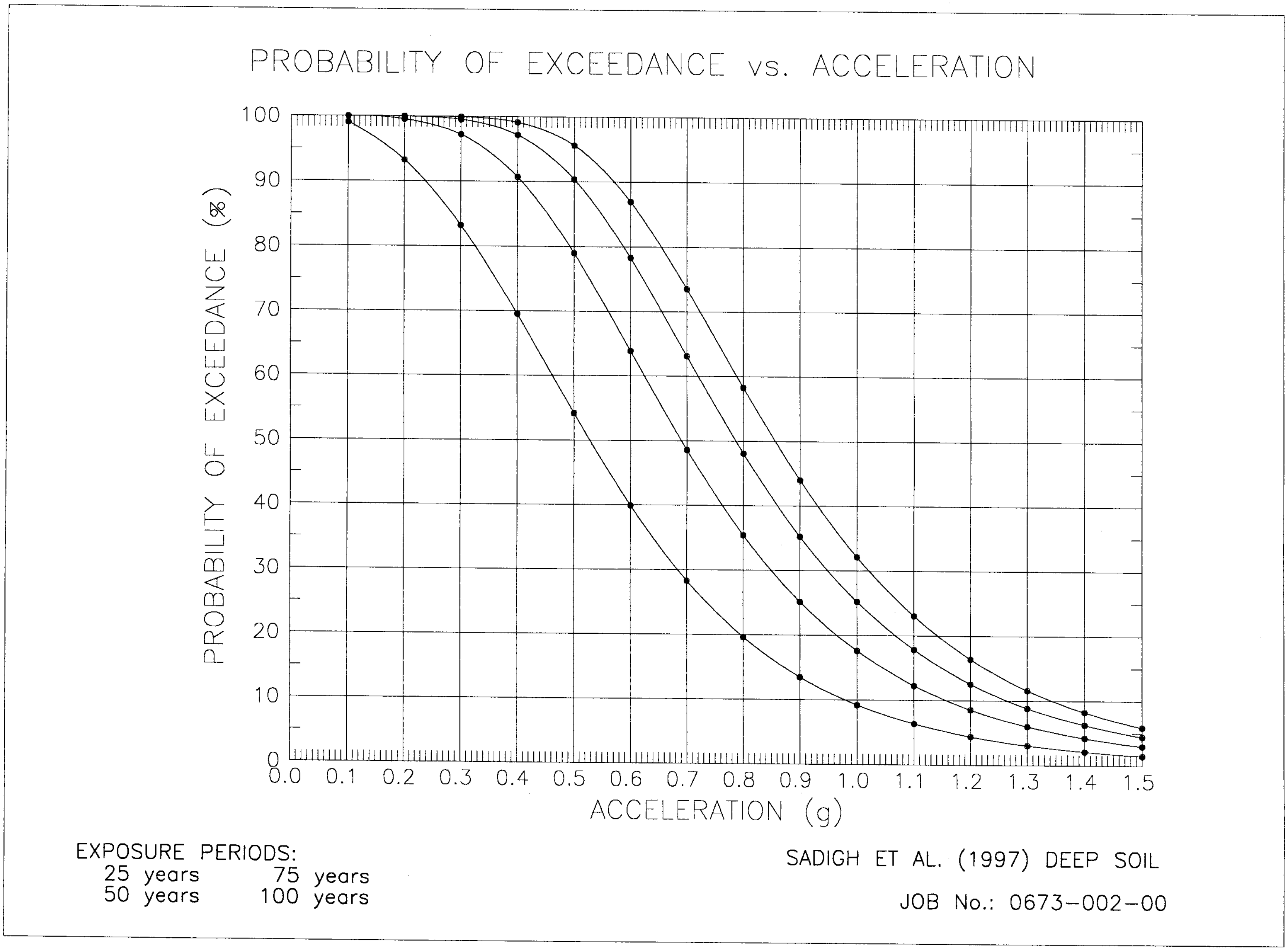


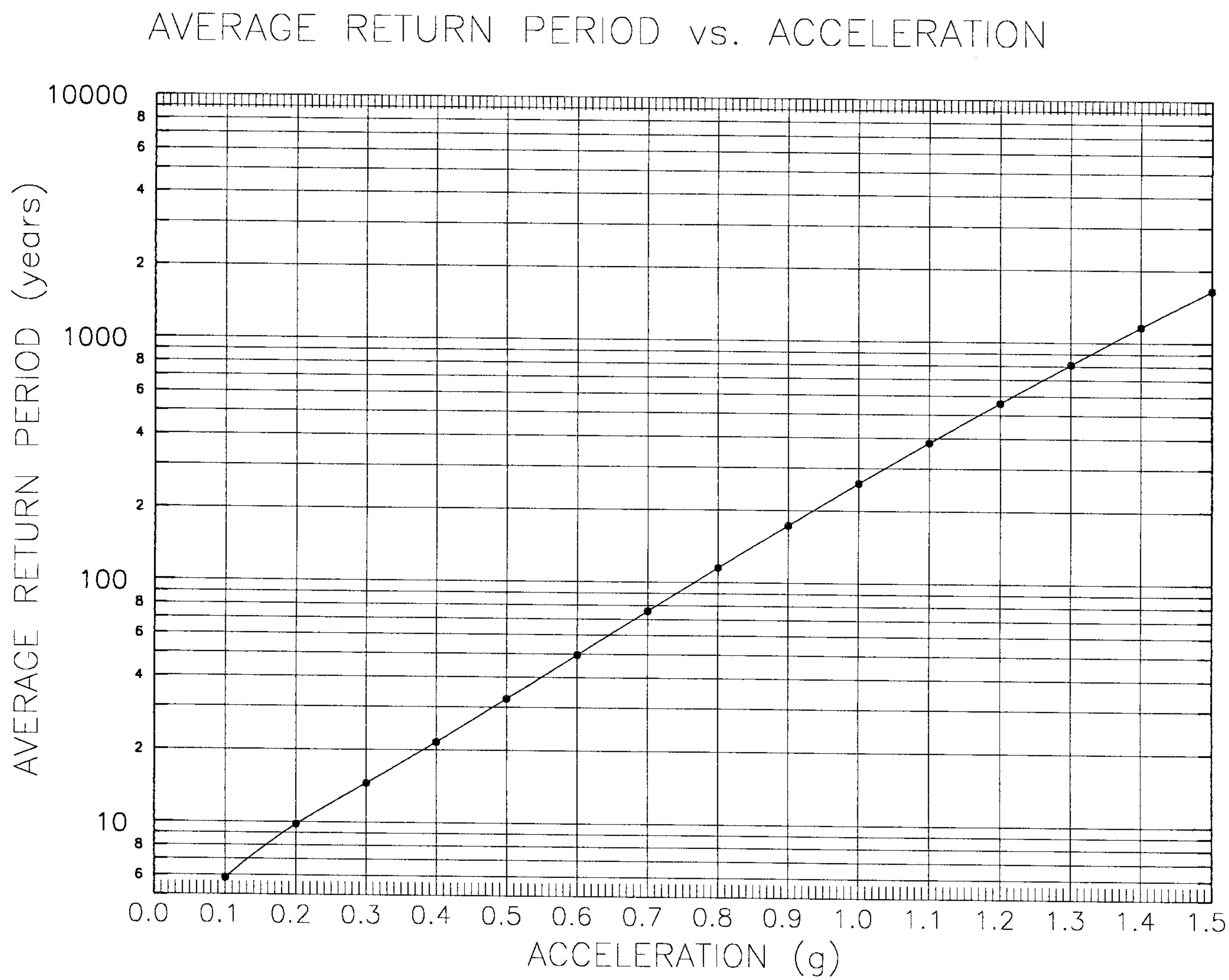
SITE LOCATION (+):

Latitude – 33.1680 N
Longitude – 115.6190 W
CALENERGY GEOTHERMAL PLANT NO. 6

FRISKSP FAULT MAP

JOB No.: 0673-002-00

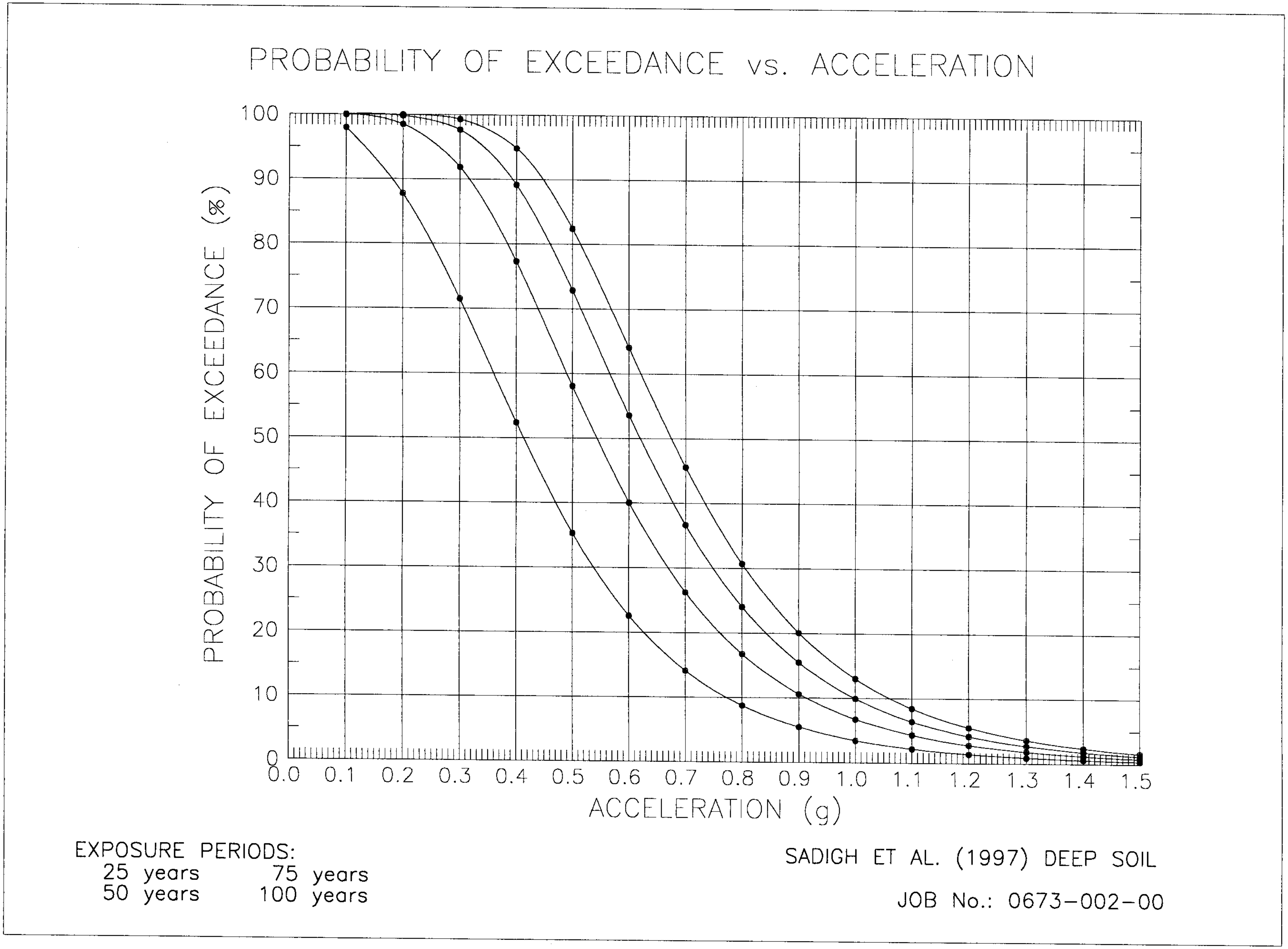

PROBABILITY OF EXCEEDANCE vs. ACCELERATION
PROBABILITY OF EXCEEDANCE (%)
100
90
80
70
60
50
40
30
20
10
0
0.0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0 1.1 1.2 1.3 1.4 1.5
ACCELERATION (g)
EXPOSURE PERIODS:
25 years
50 years
75 years
100 years
SADIGH ET AL. (1997) DEEP SOIL
JOB No.: 0673-002-00

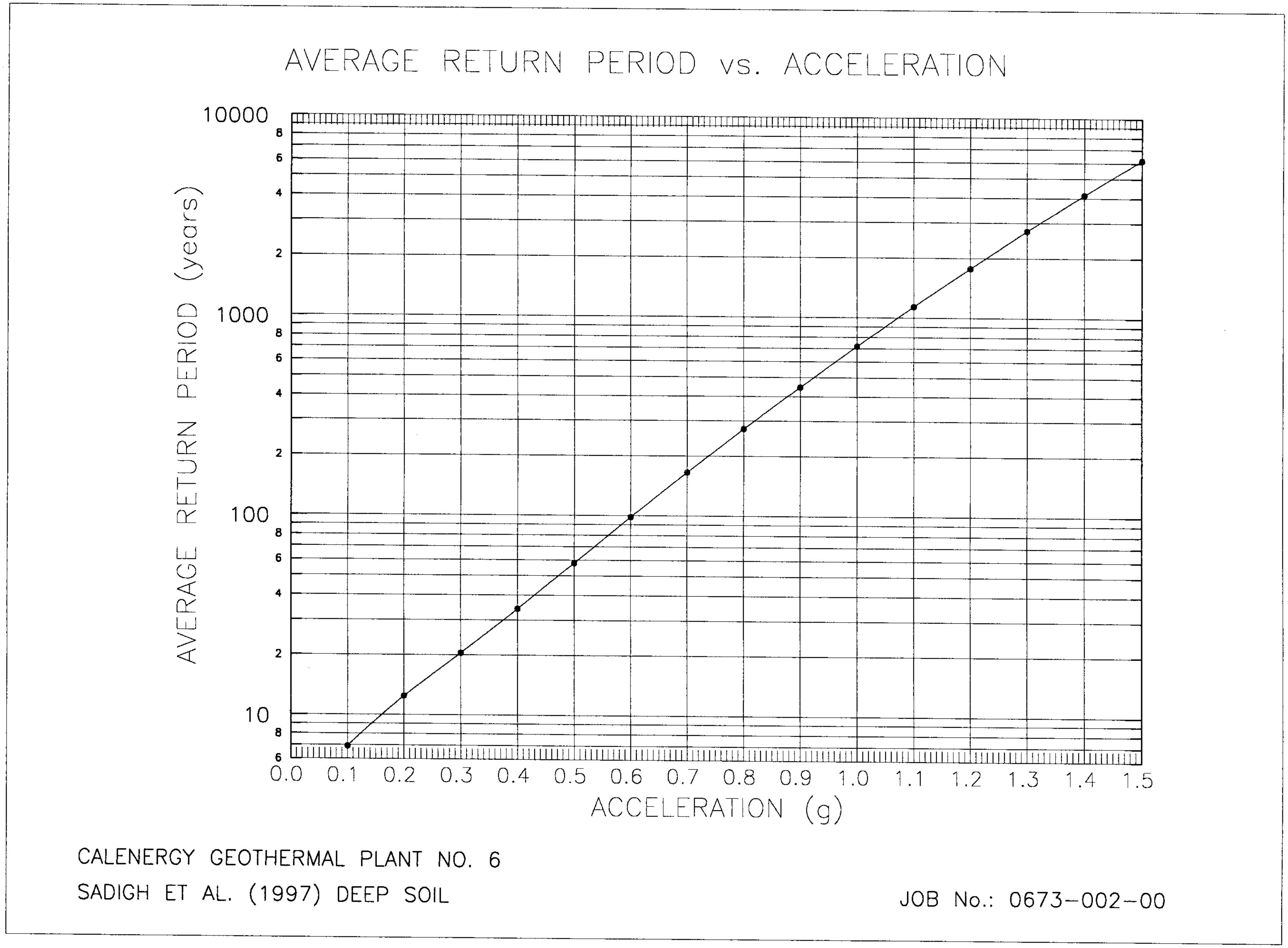


CALENERGY GEOTHERMAL PLANT NO. 6

SADIGH ET AL. (1997) DEEP SOIL

JOB No.: 0673-002-00


PROBABILITY OF EXCEEDANCE vs. ACCELERATION
PROBABILITY OF EXCEEDANCE (%)
100
90
80
70
60
50
40
30
20
10
0
0.0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0 1.1 1.2 1.3 1.4 1.5
ACCELERATION (g)
EXPOSURE PERIODS:
25 years
50 years
75 years
100 years
SADIGH ET AL. (1997) DEEP SOIL
JOB No.: 0673-002-00


AVERAGE RETURN PERIOD vs. ACCELERATION
AVERAGE RETURN PERIOD (years)
10000
1000
100
10
8
6
4
2
0.0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0 1.1 1.2 1.3 1.4 1.5
ACCELERATION (g)
CALENERGY GEOTHERMAL PLANT NO. 6
SADIGH ET AL. (1997) DEEP SOIL
JOB No.: 0673-002-00

APPENDIX E

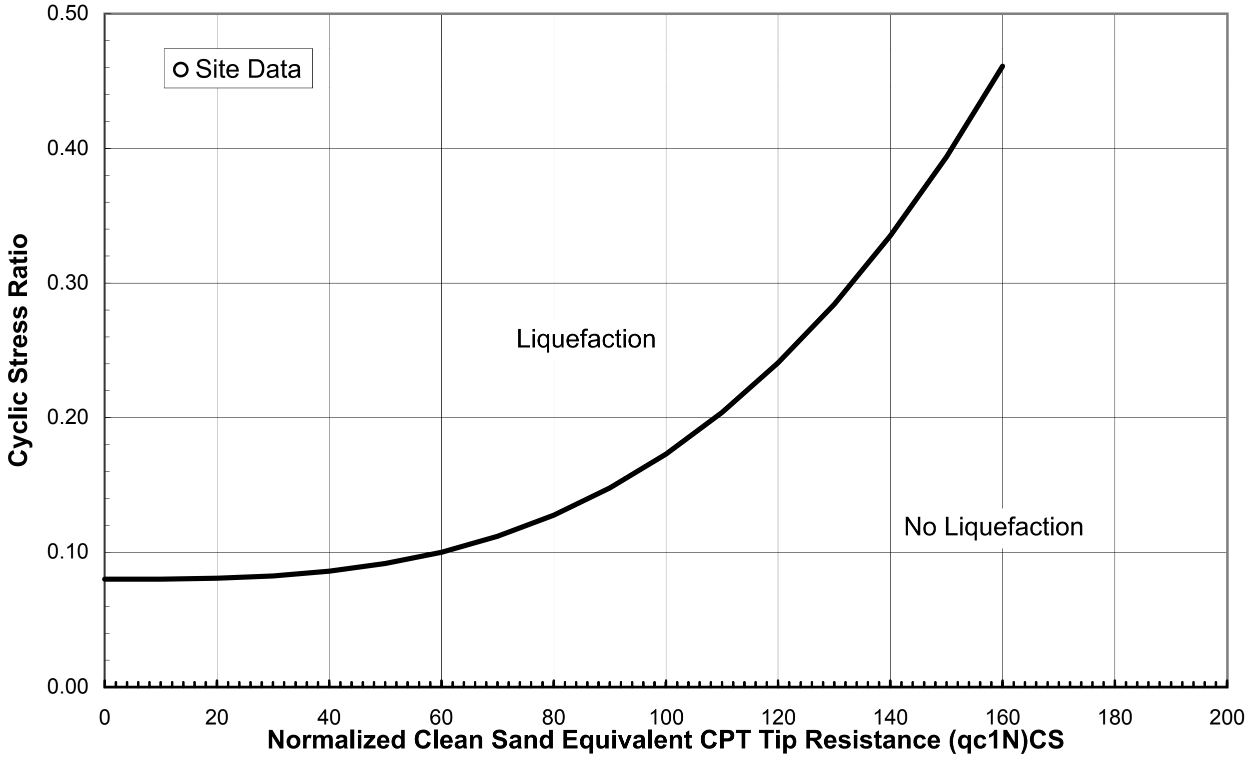
LIQUEFACTION ANALYSIS

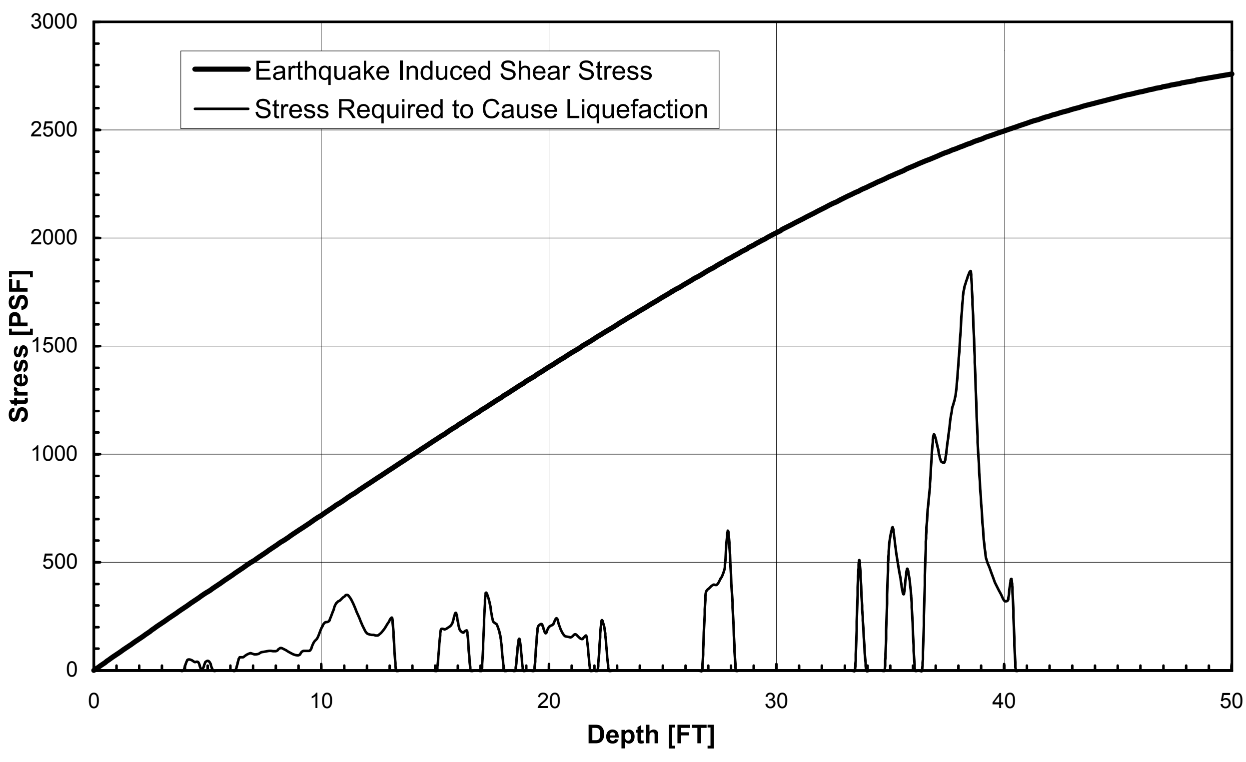
Liquefaction analysis was performed on the data gathered from the CPT soundings. The liquefaction analysis was based on the simplified techniques originally presented by Seed and Idriss (1982), with recent improvements from the 1996 and 1998 NCEER workshops as summarized by Youd et al (2001). The liquefaction analysis was conducted in general accordance with the recommended procedures for implementation of DMG special publication 117 (SCES, 1999). The CPT data was normalized for overburden pressure, and corrected for fines content and thin layers using the methods described in the referenced document (Youd et al, 2001). The CPT fines correction was based on the soil behavior type index (Ic). Note that the CPT data gathered above the groundwater table, or in clayey soils was not included in the analysis, because these materials are not considered to be susceptible to liquefaction.

The results of the liquefaction analyses for CPT soundings 1 through 9 are presented in Figures E-1.1 through E-9.3, respectively. The top chart in the first figure for each CPT sounding shows the relationship used between the cyclic resistance ratio (CRR) and the in-situ soil index (normalized clean sand equivalent CPT tip resistance $(q_{c1N})_{cs}$ for the CPT data). Also shown on this chart are the CPT data obtained during the field investigation. The lower chart in the first figure for each sounding shows the average shear stresses induced in the soil profile due to an event with a peak ground acceleration of 0.46g, as well as the shear stress required to cause liquefaction in the soil profile. Both of these parameters are plotted as a function of depth. In all areas where the earthquake induced shear stress exceeds the stress required to cause liquefaction, liquefaction is possible.

A magnitude weighted peak ground acceleration (PGA) of 0.46g corresponds roughly to the highest *median* deterministic PGA estimated using the attenuation relationships for deep soil sites (Sadigh, 1997). According to the probabilistic analysis, this PGA also has a magnitude weighted return period of roughly 50 years. The probabilistic analysis suggests that the design basis magnitude weighted PGA with a 10 percent probability of being exceeded in 50 years is about 0.92g. The second figure for each CPT sounding presents the results of the liquefaction analyses assuming PGA's of 0.1g, 0.2g, 0.4g and 0.8g. These charts give an idea of the return period of liquefaction at the site.

The third chart for each CPT sounding presents the apparent soil classification used for the analysis. Note that this classification is a function of the normalized cone resistance (Q) and the normalized friction ratio (F). The CPT data suggests that the site is underlain by interbedded silty sands (SM), sandy silts (ML) and clays (CL). The laboratory gradation analyses provided a reasonably good correlation with the soil classifications determined by the CPT analyses.





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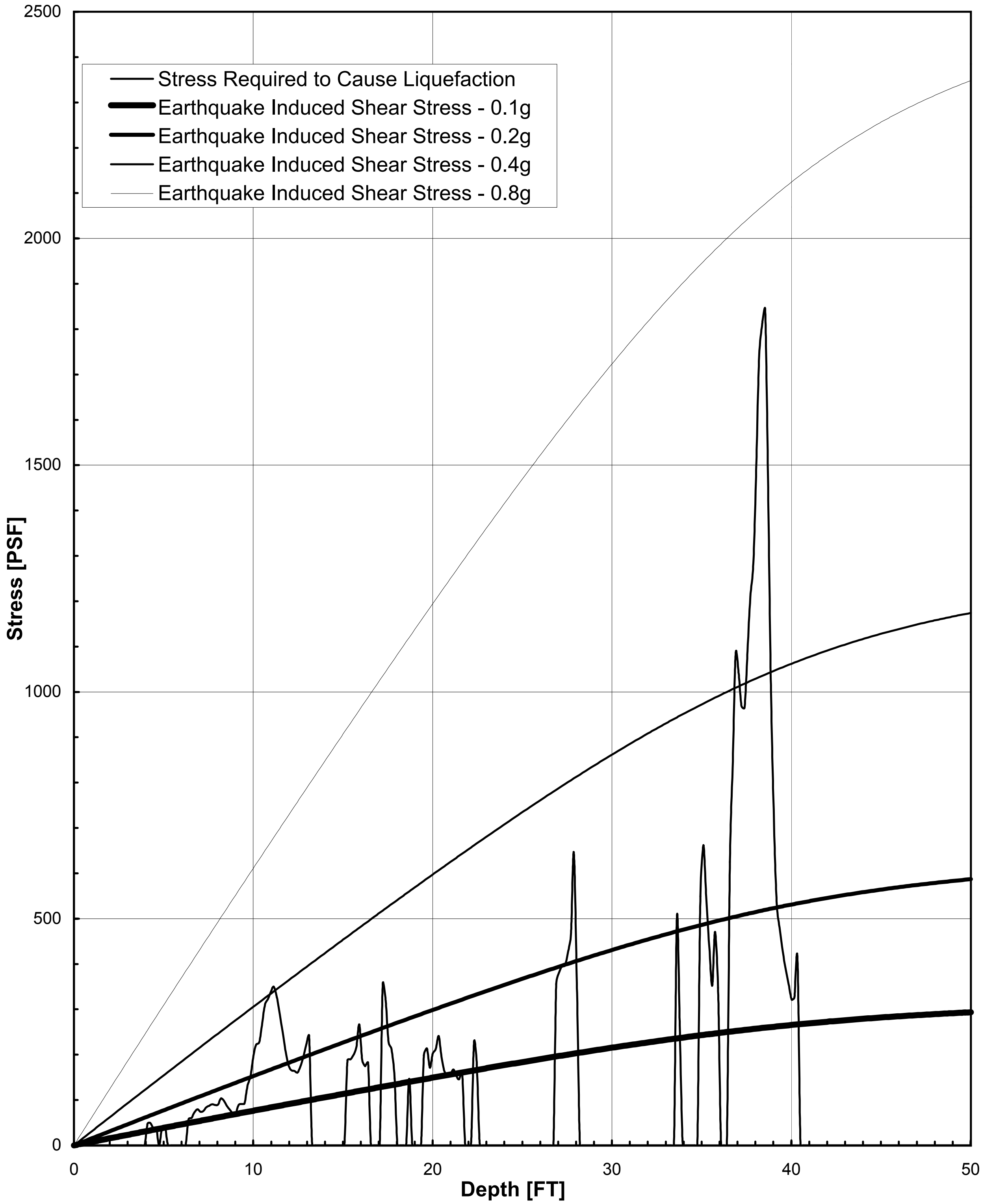
Liquefaction Analysis (CPT-1)

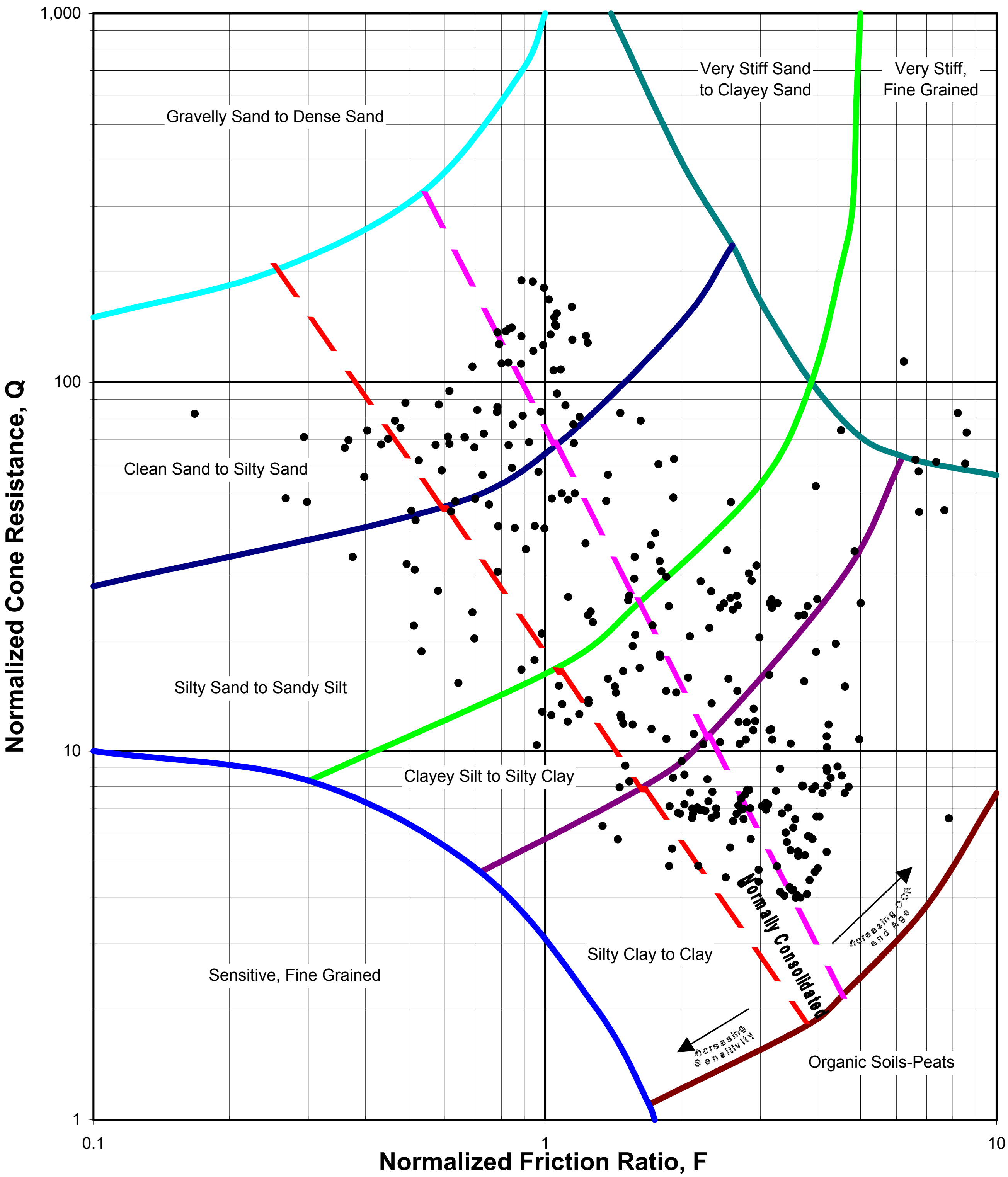
Project No. 0673-002-00
Document No. 02-0022
FIGURE E-1.1



Geotechnics Incorporated

Liquefaction Analysis (CPT-1)

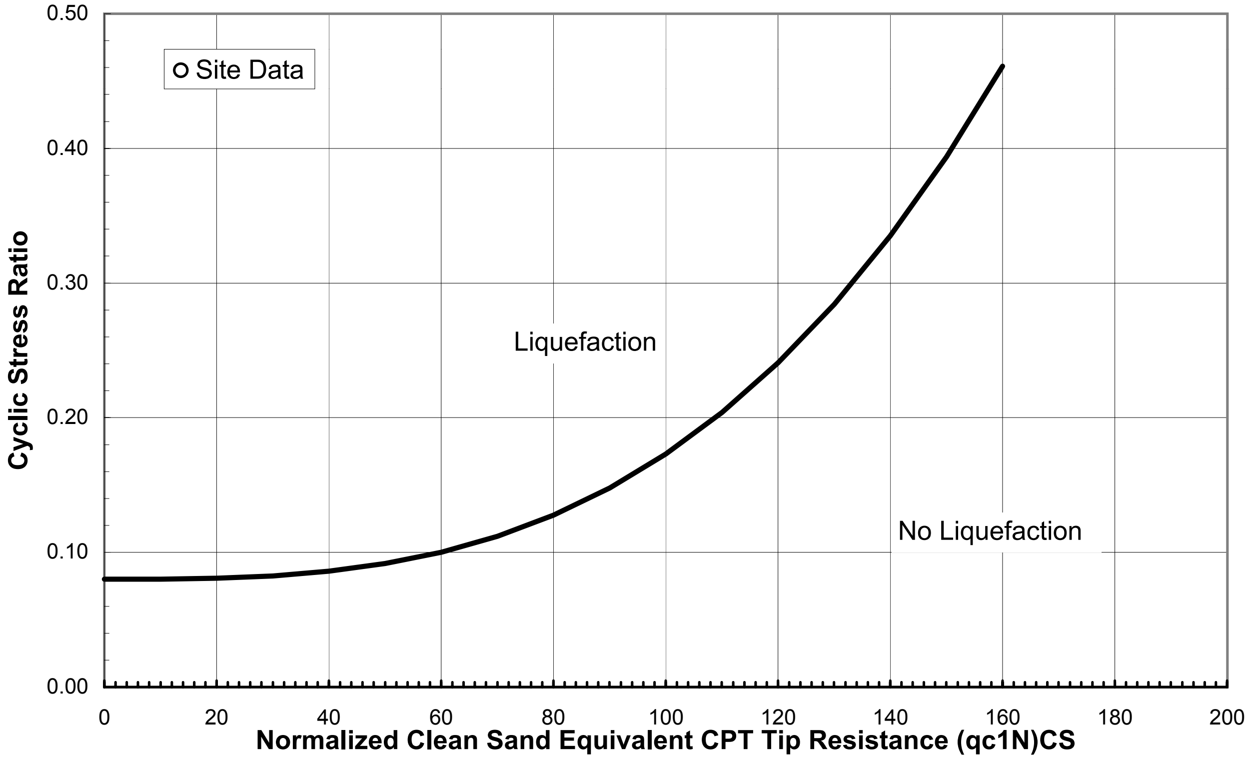
Project No. 0673-002-00
Document No. 02-0022
FIGURE E-1.2

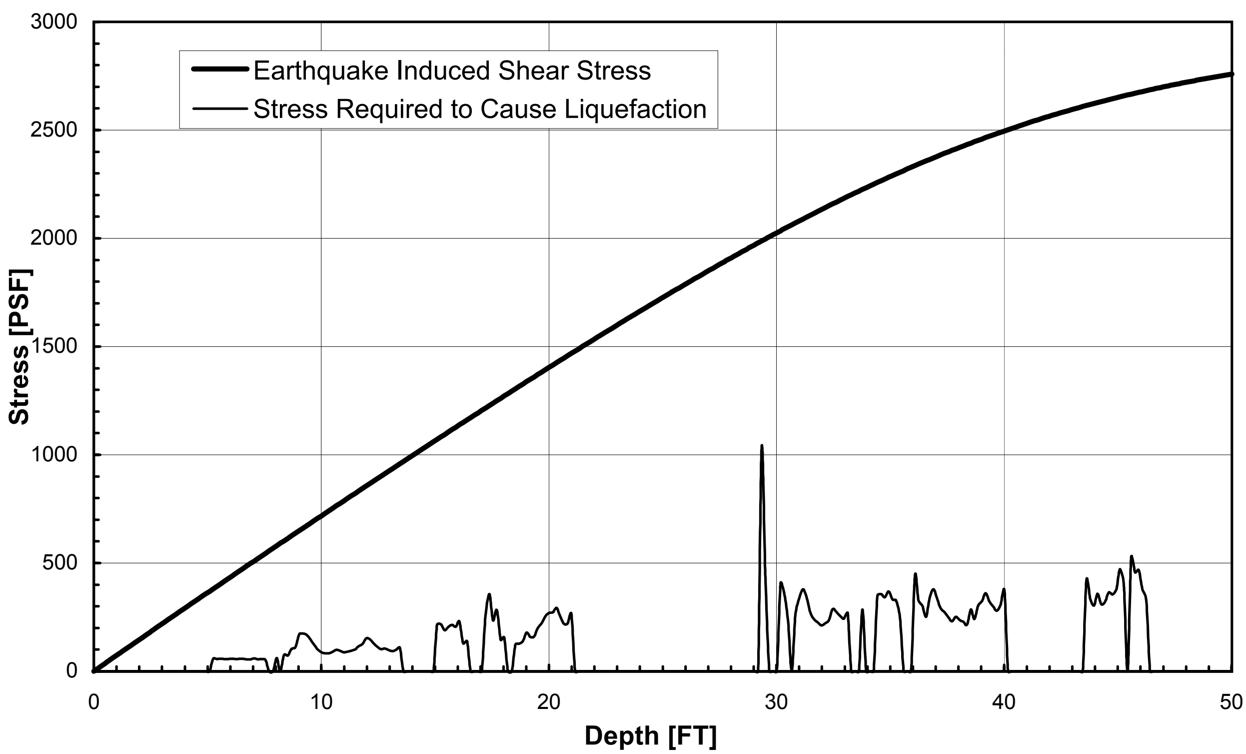


Geotechnics Incorporated

Soil Classification (CPT-1)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-1.3





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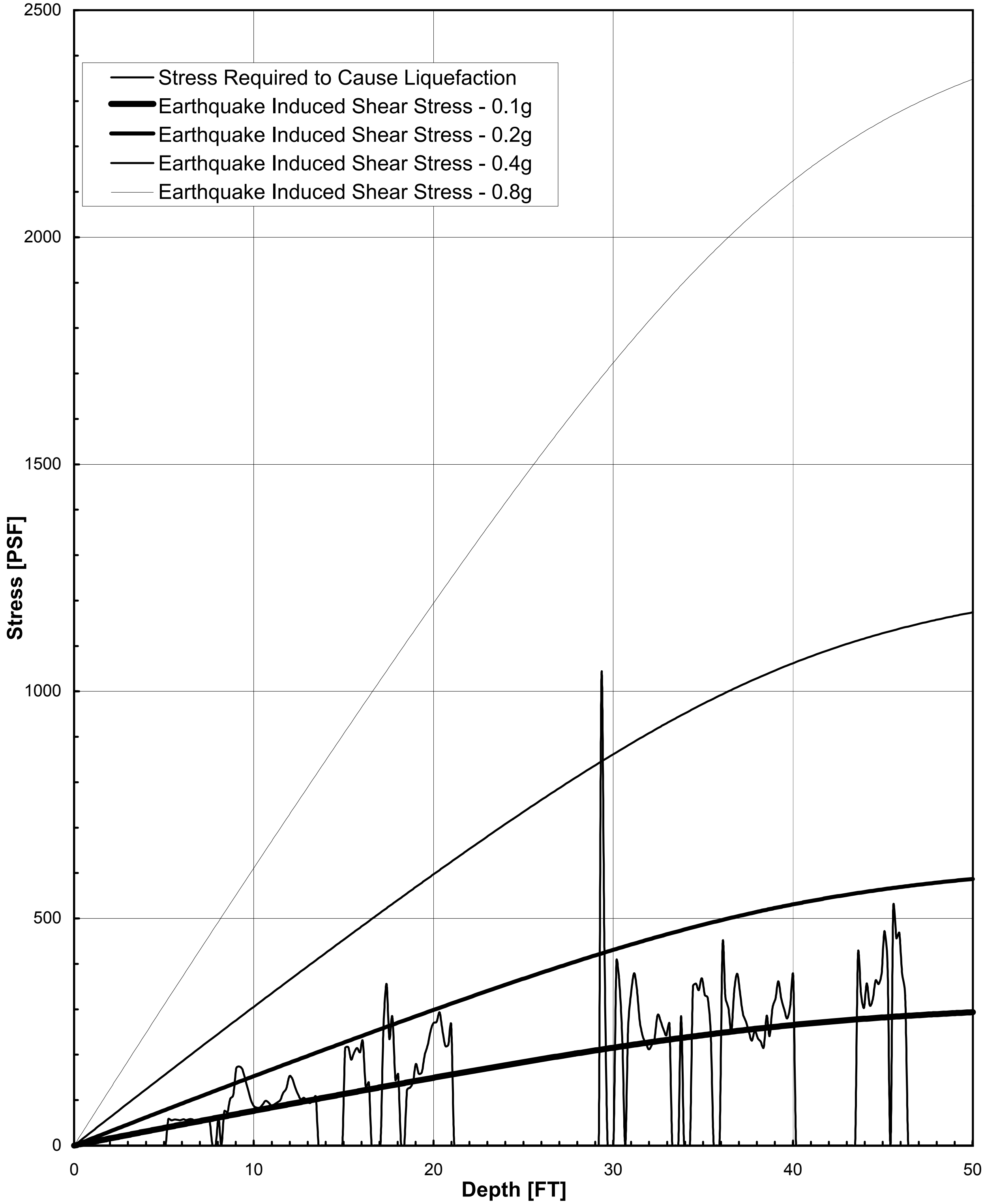
Liquefaction Analysis (CPT-2)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-2.1



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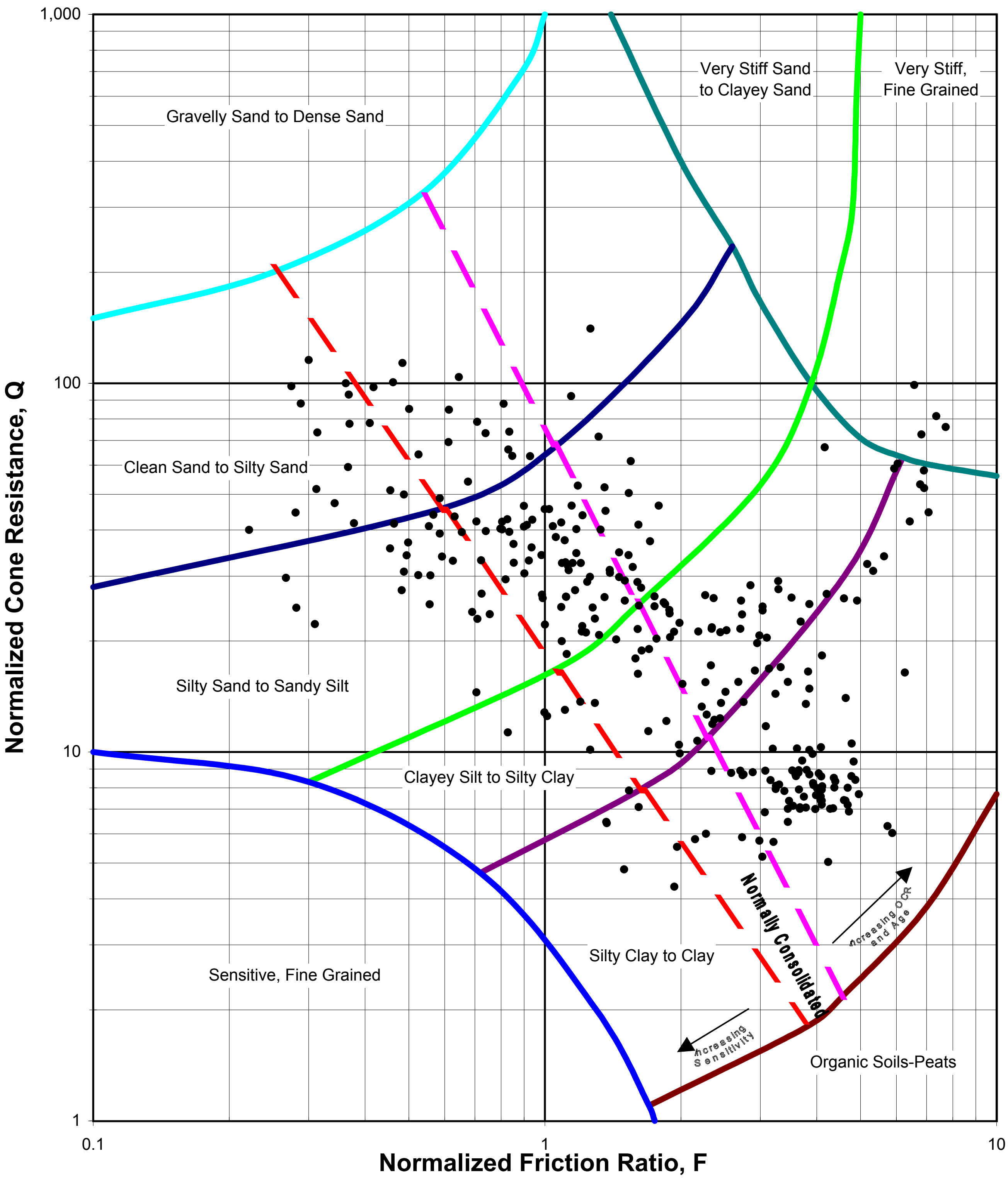
Liquefaction Analysis (CPT-2)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-2.2



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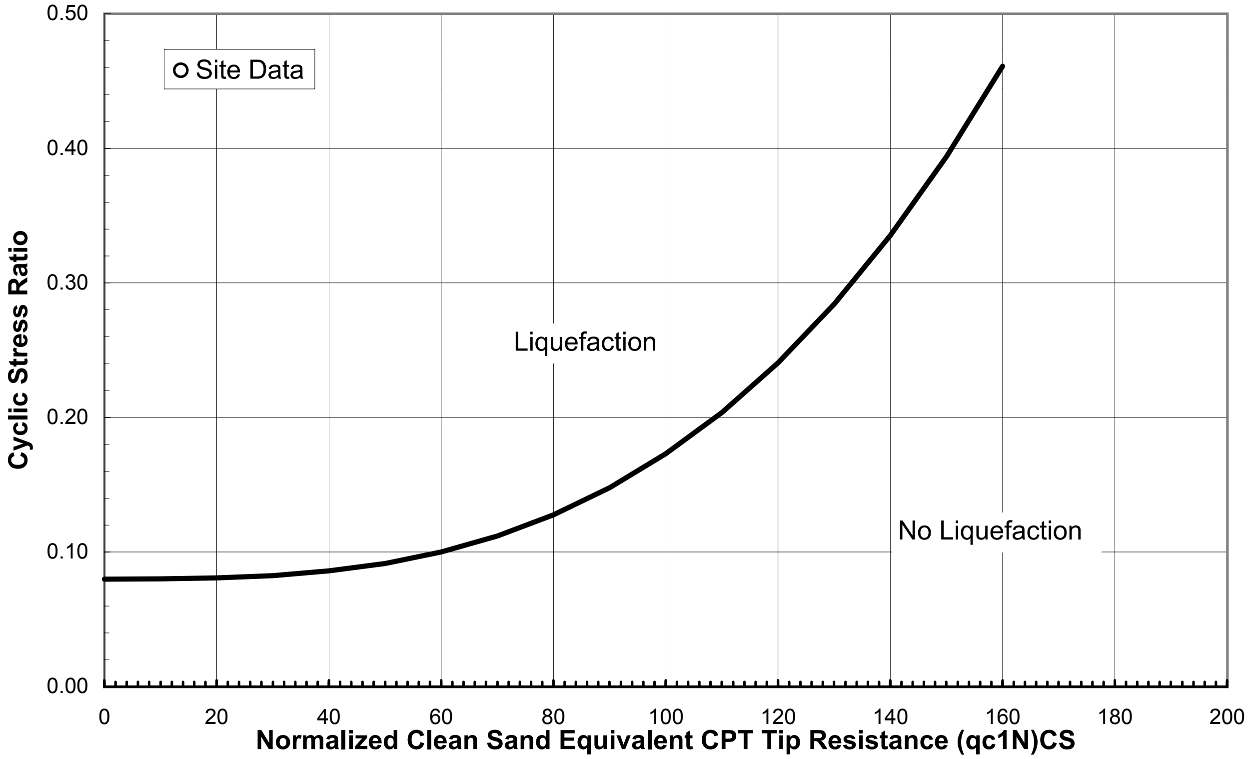
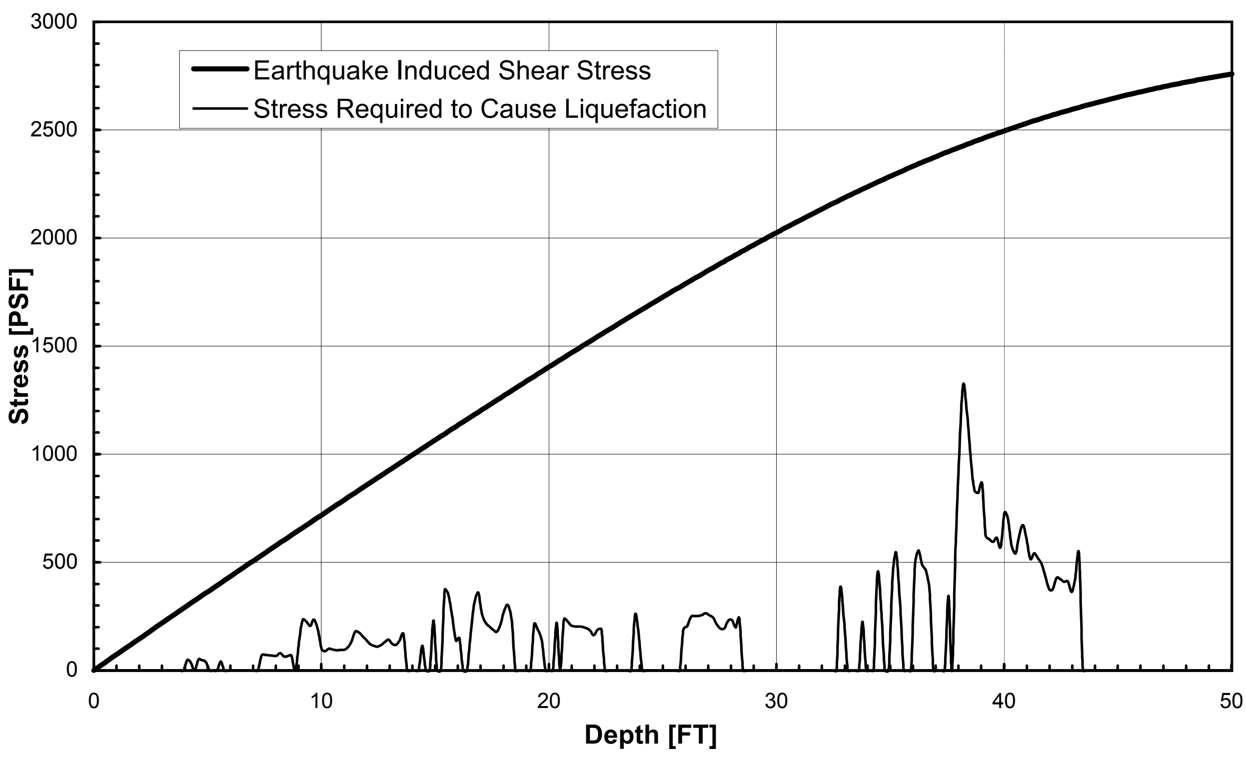
Soil Classification (CPT-2)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-2.3

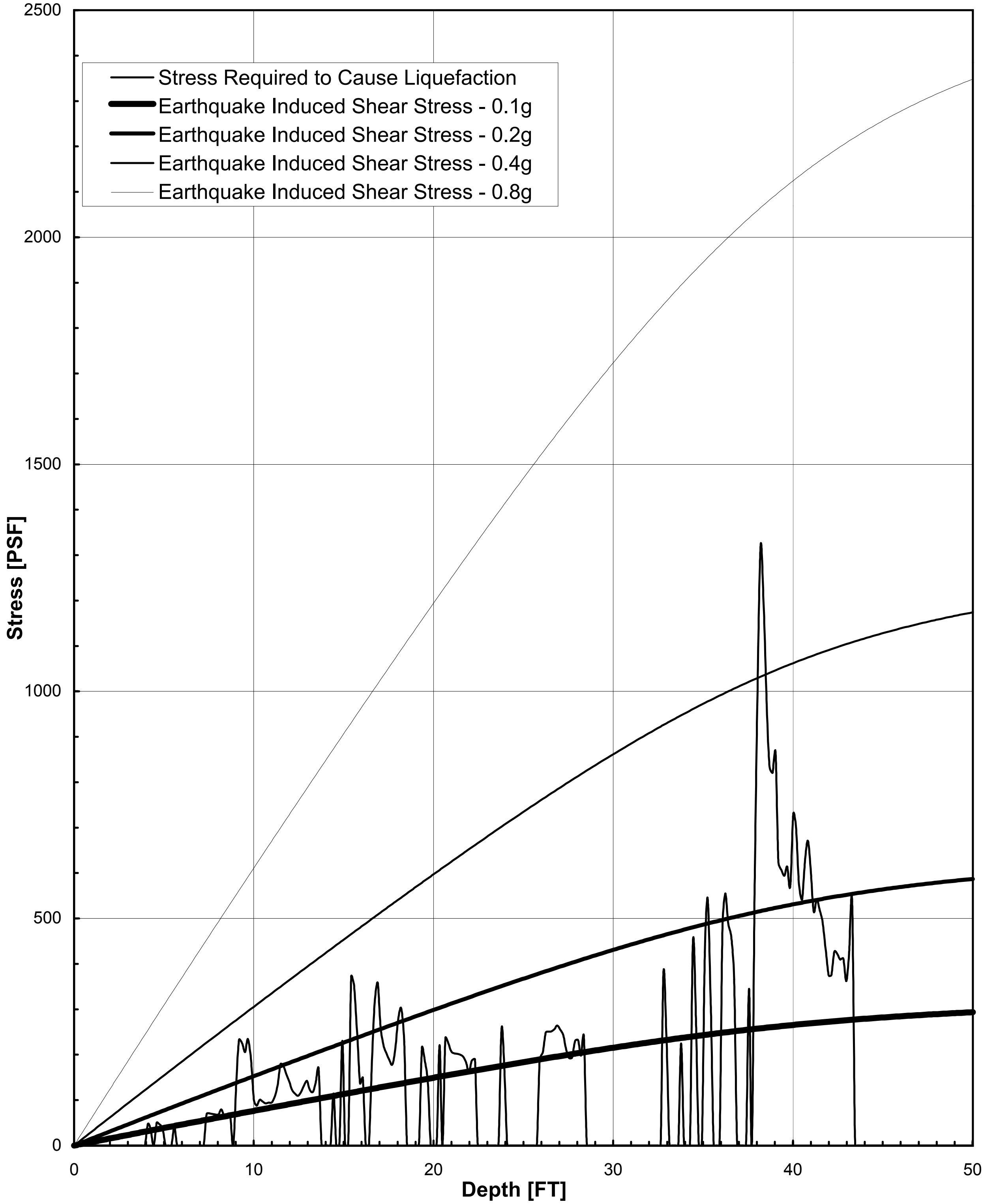




Geotechnics Incorporated

Liquefaction Analysis (CPT-3)

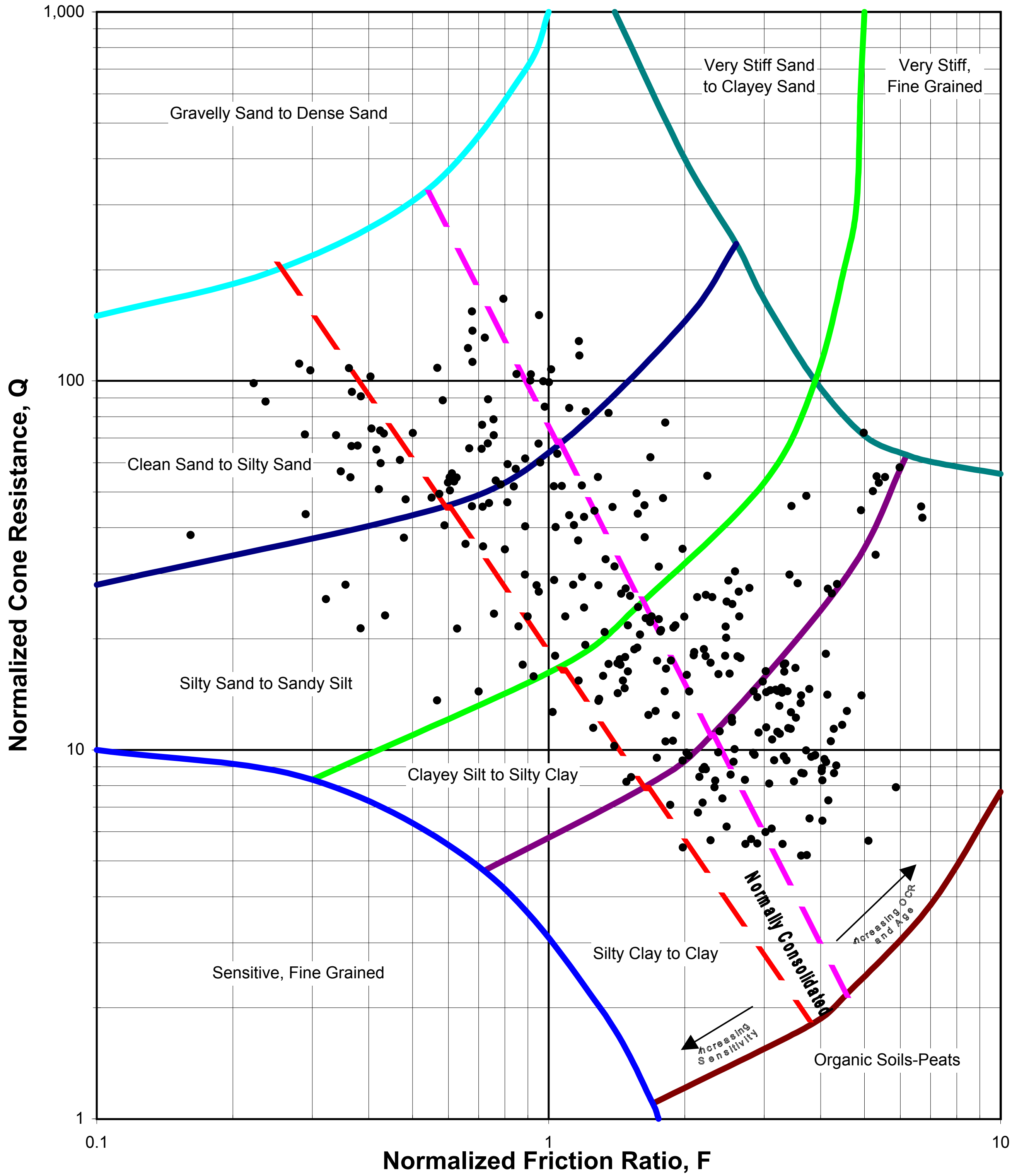
Project No. 0673-002-00
Document No. 02-0022
FIGURE E-3.1



Geotechnics Incorporated

Liquefaction Analysis (CPT-3)

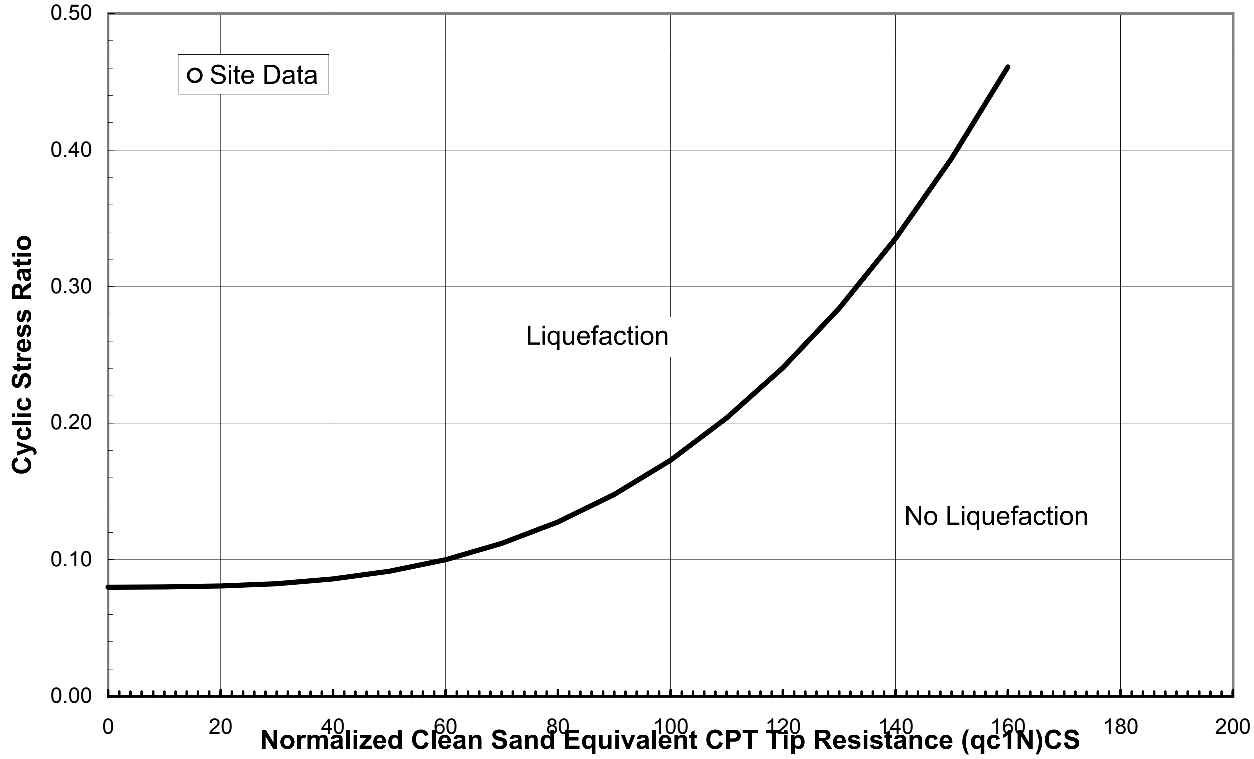
Project No. 0673-002-00
Document No. 02-0022
FIGURE E-3.2

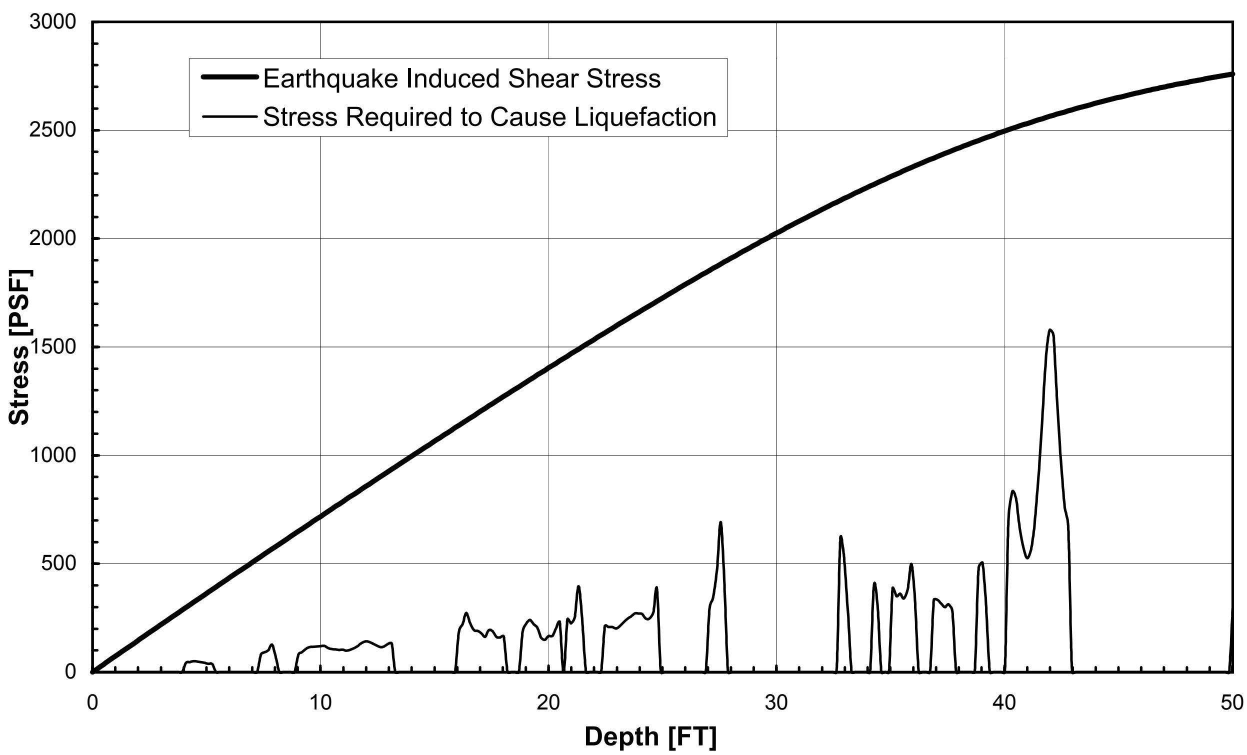


Geotechnics Incorporated

Soil Classification (CPT-3)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-3.3





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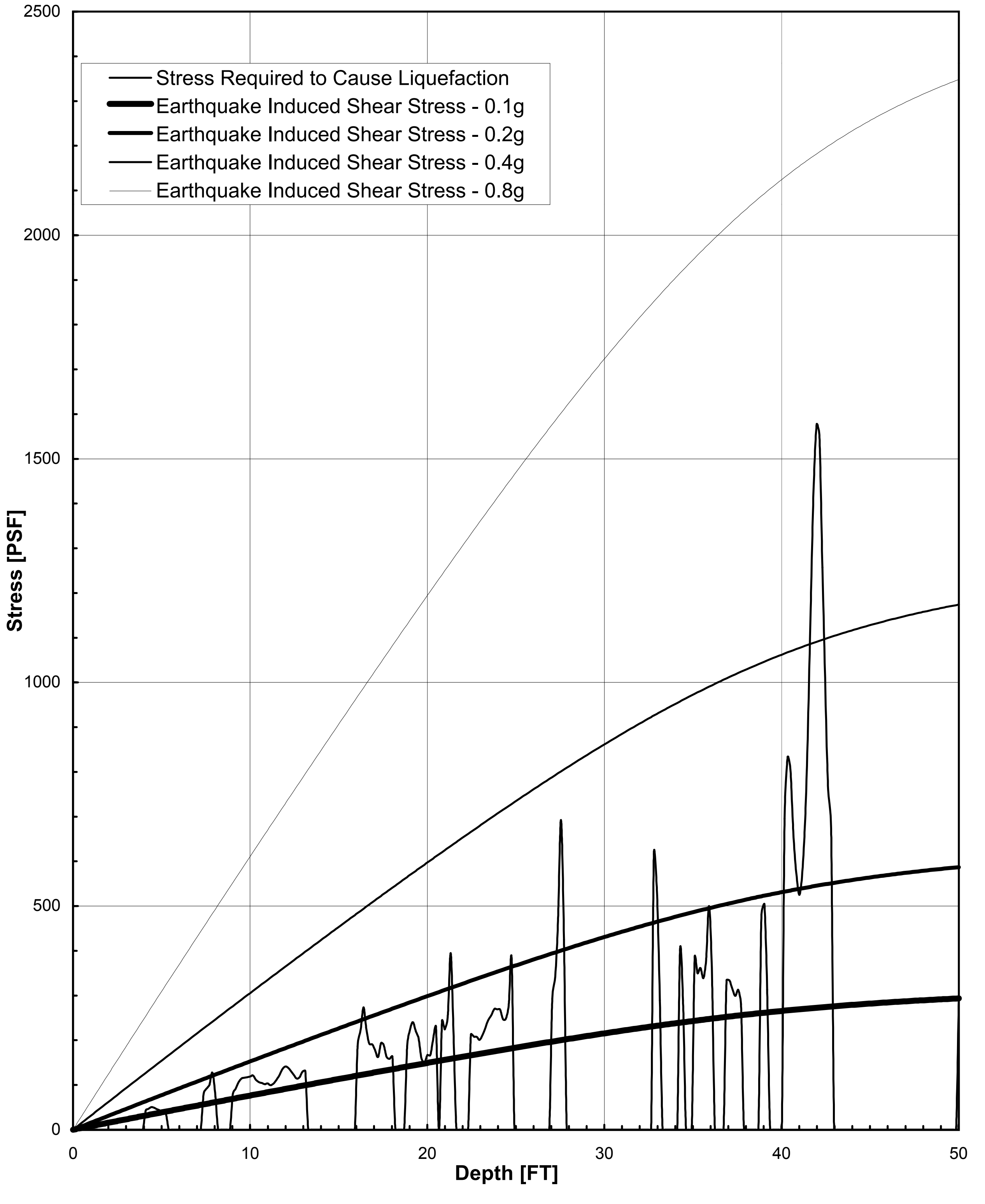
Liquefaction Analysis (CPT-4)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-4.1



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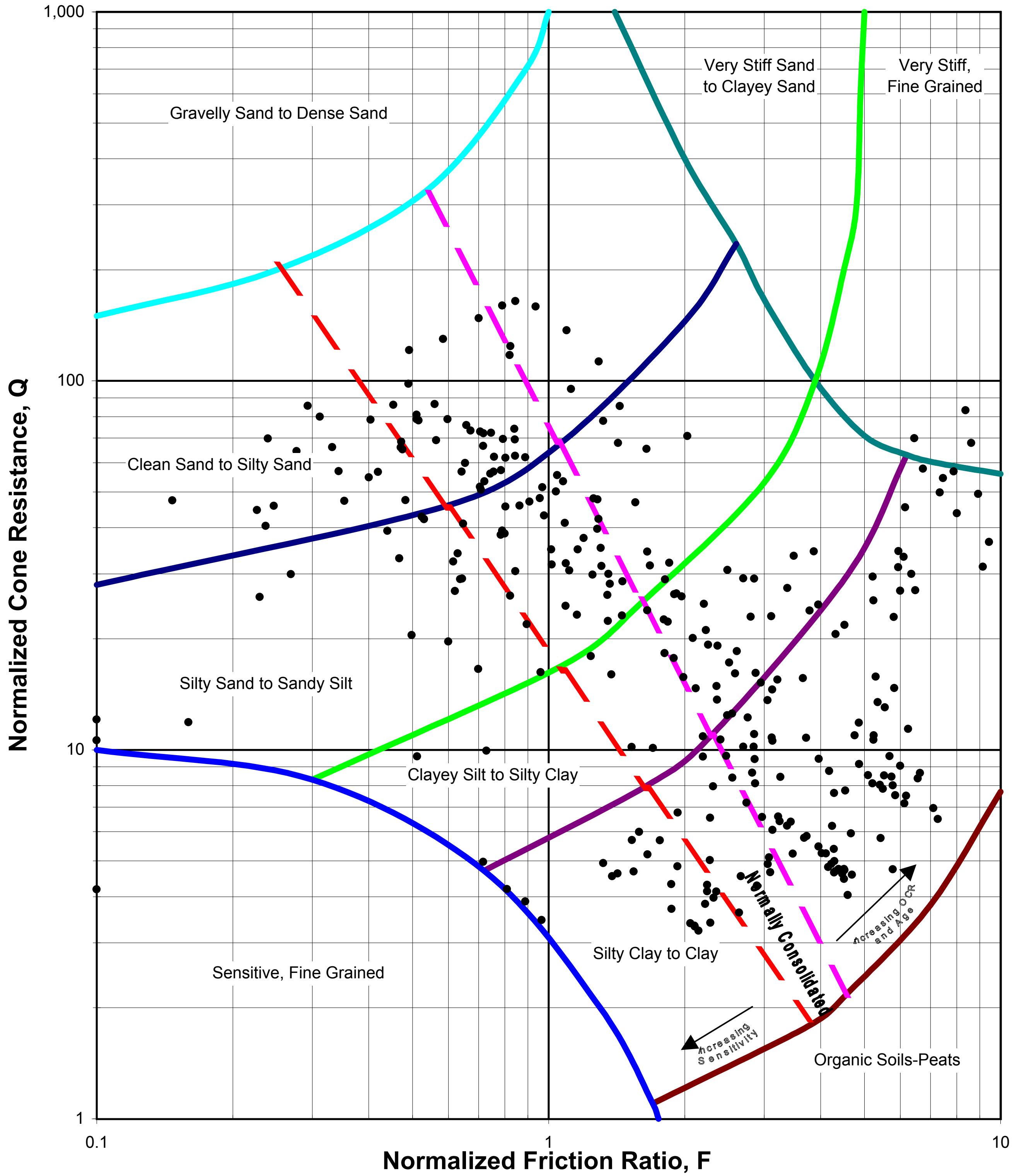
Liquefaction Analysis (CPT-4)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-4.2



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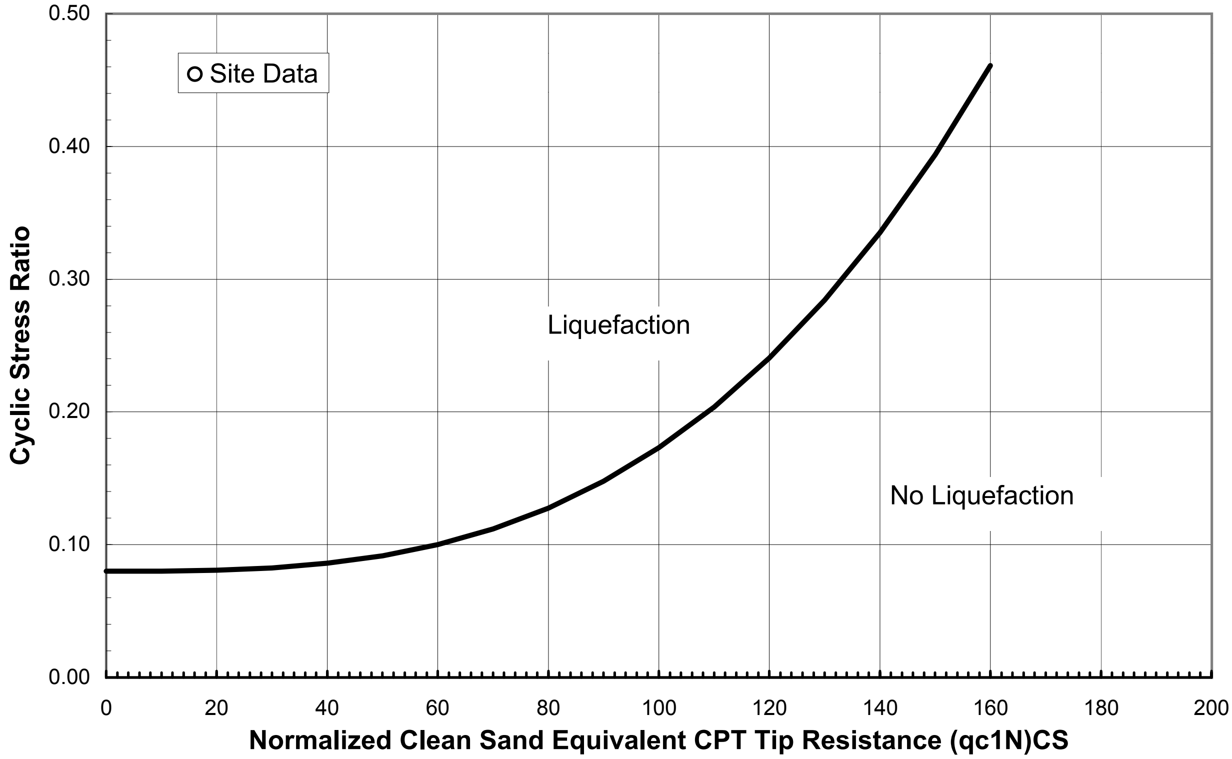
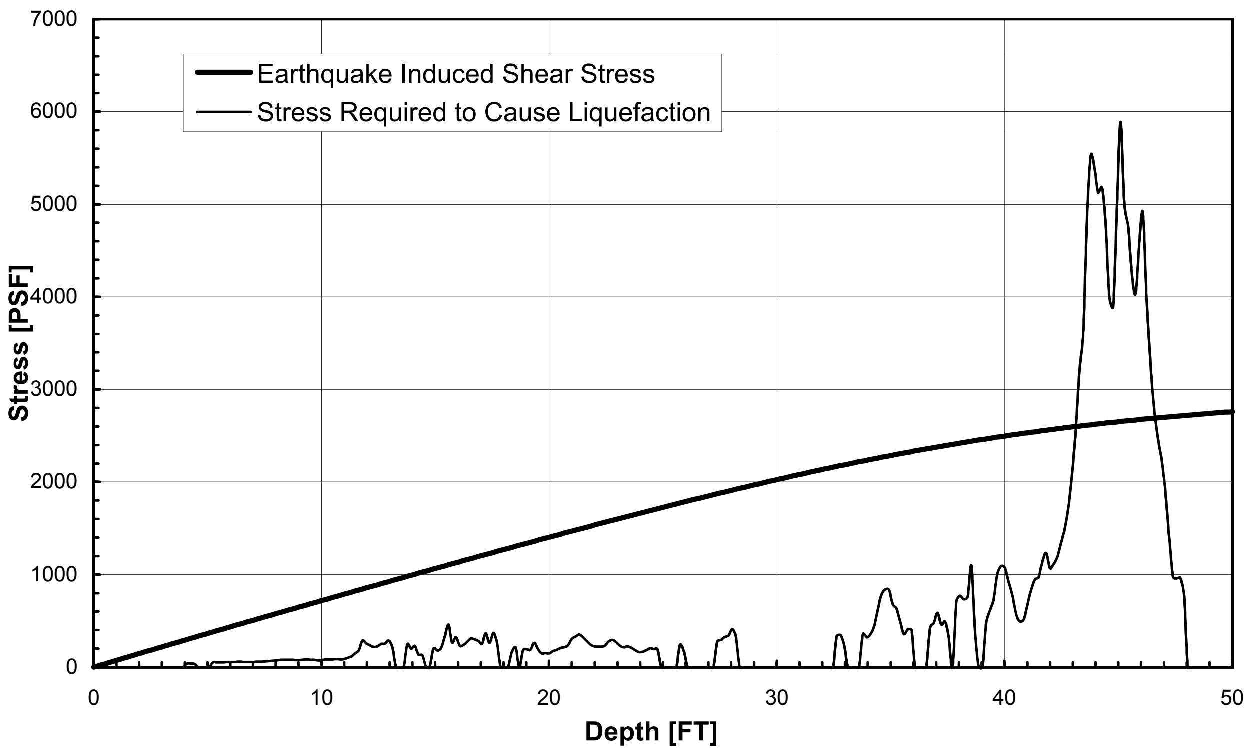
Soil Classification (CPT-4)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-4.3

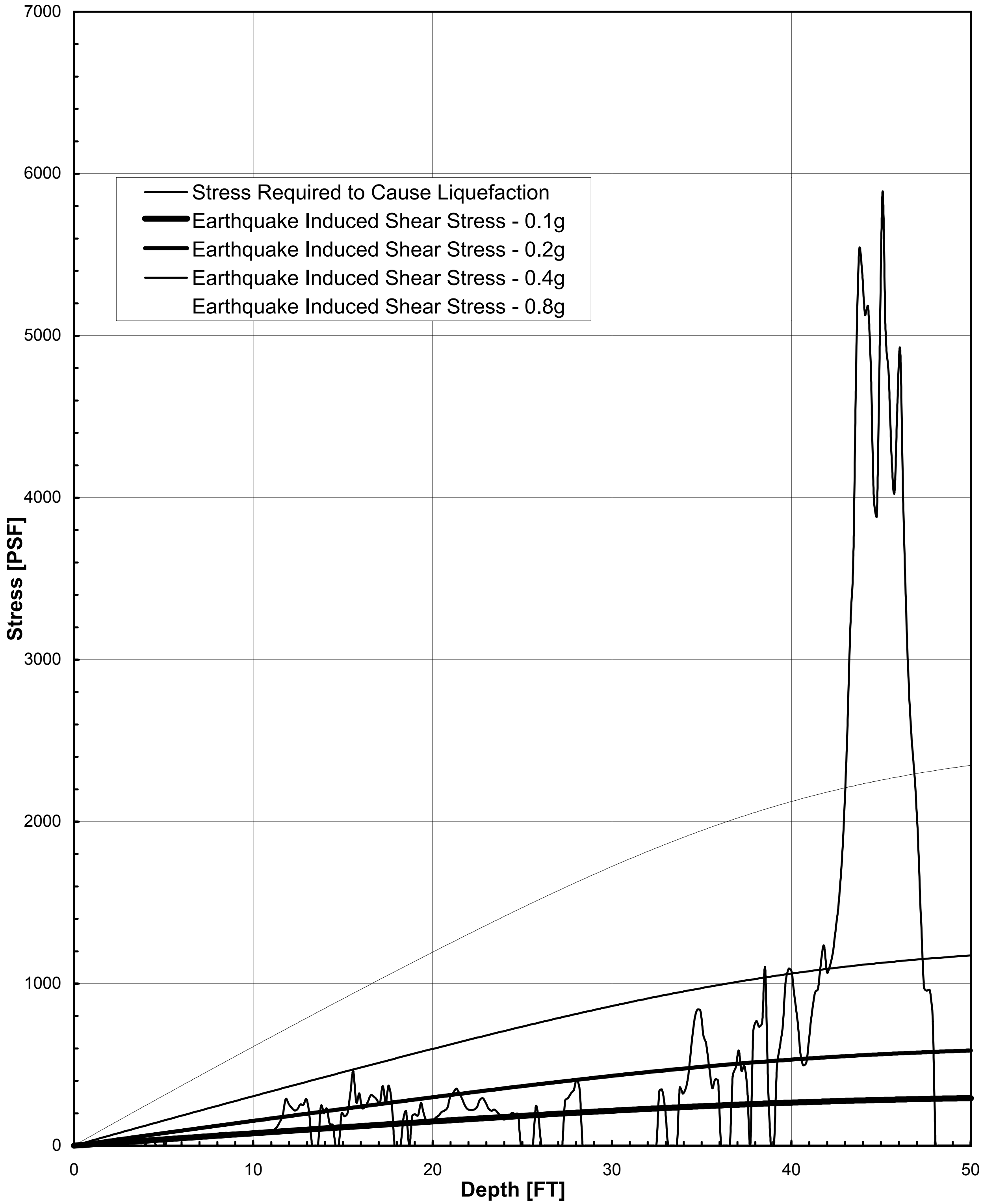




Geotechnics Incorporated

Liquefaction Analysis (CPT-5)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-5.1



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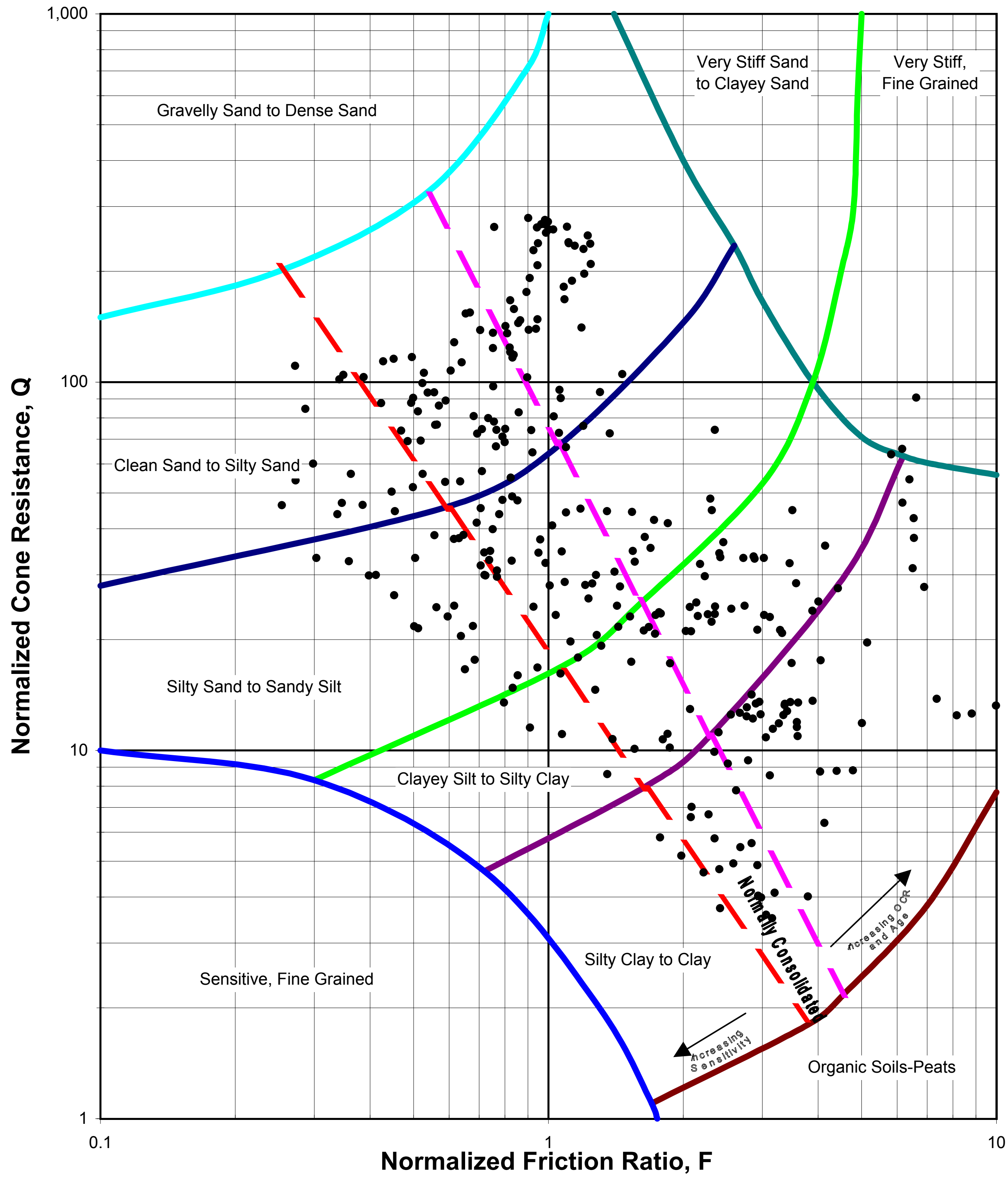
Liquefaction Analysis (CPT-5)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-5.2



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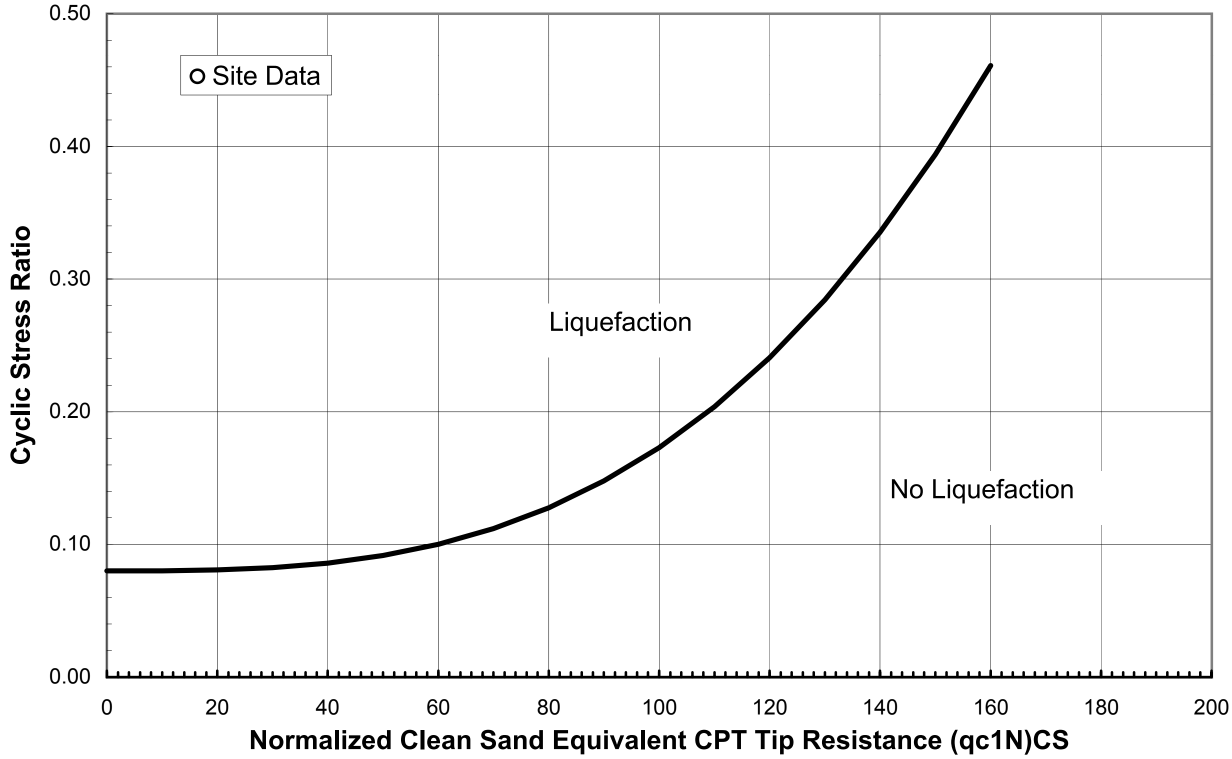
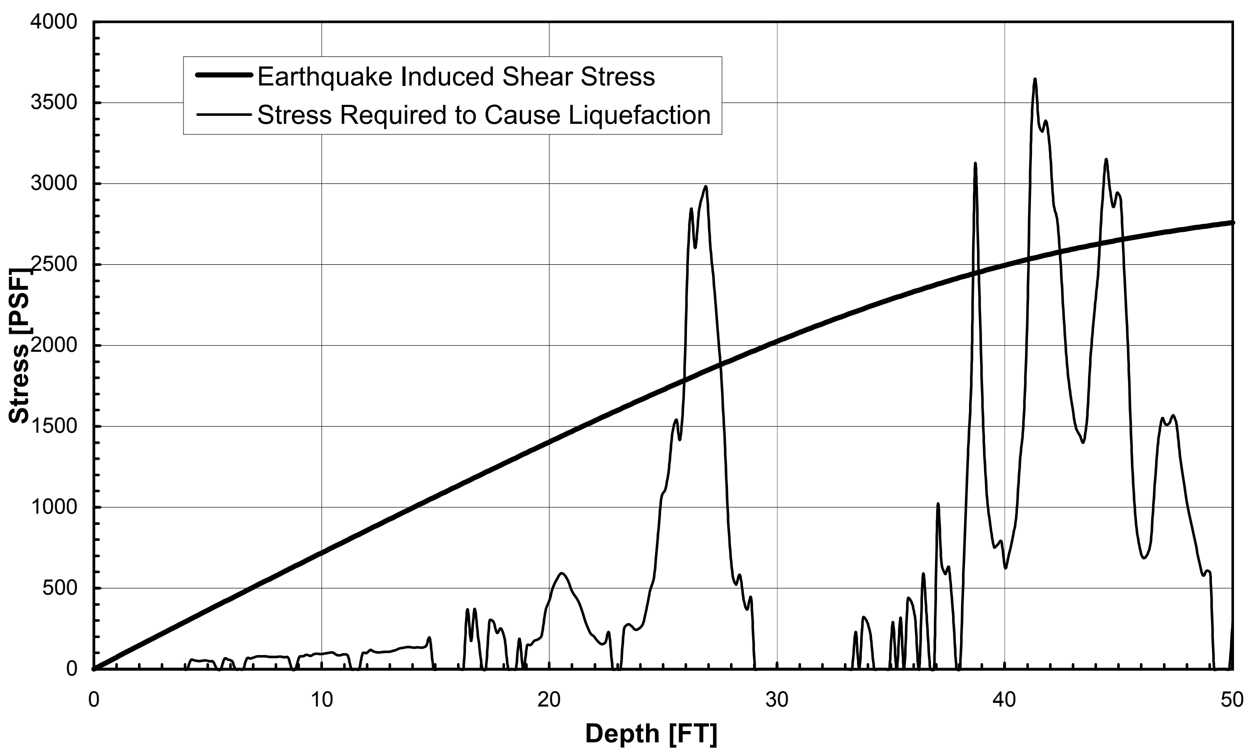
Soil Classification (CPT-5)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-5.3





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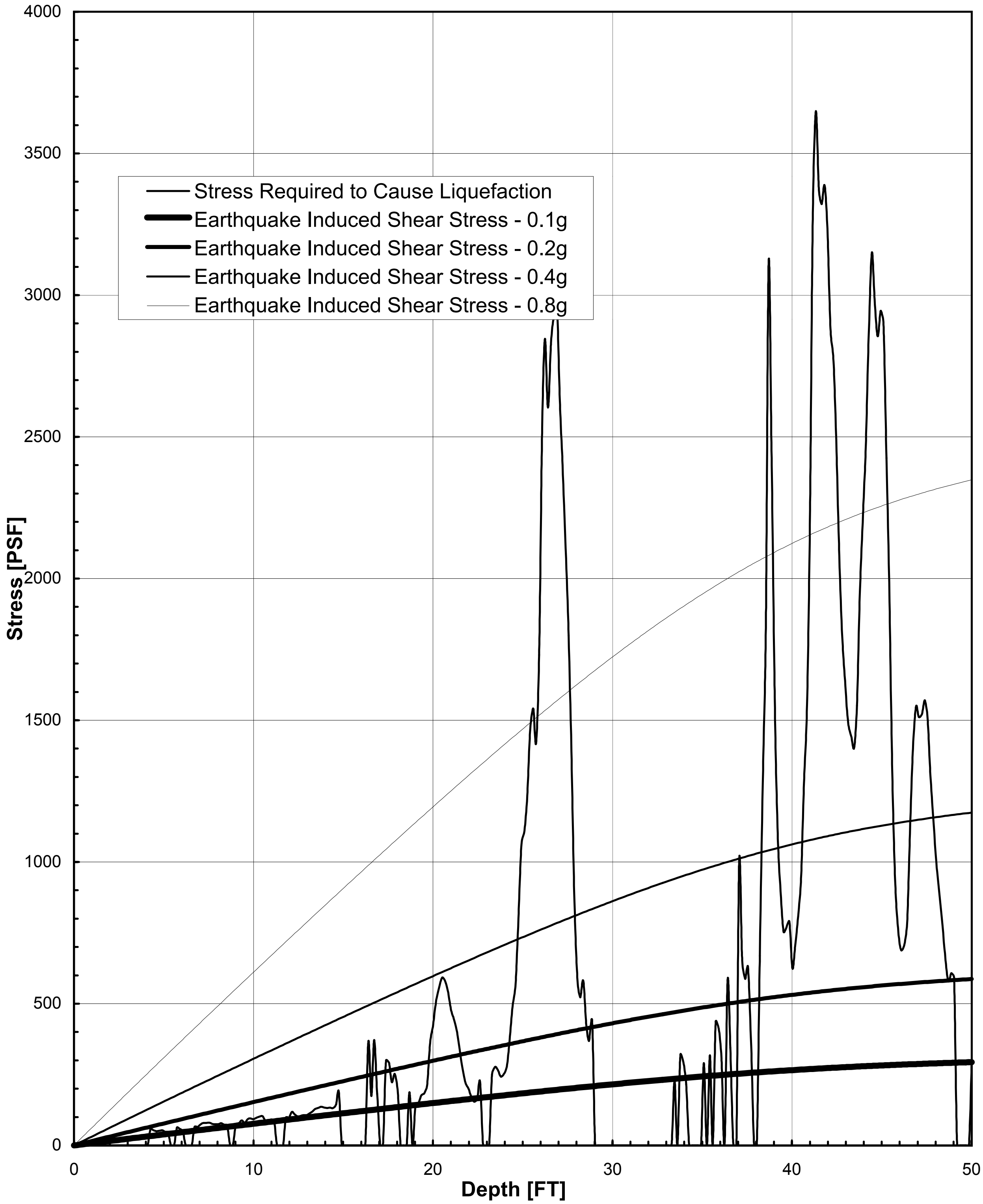
Liquefaction Analysis (CPT-6)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-6.1



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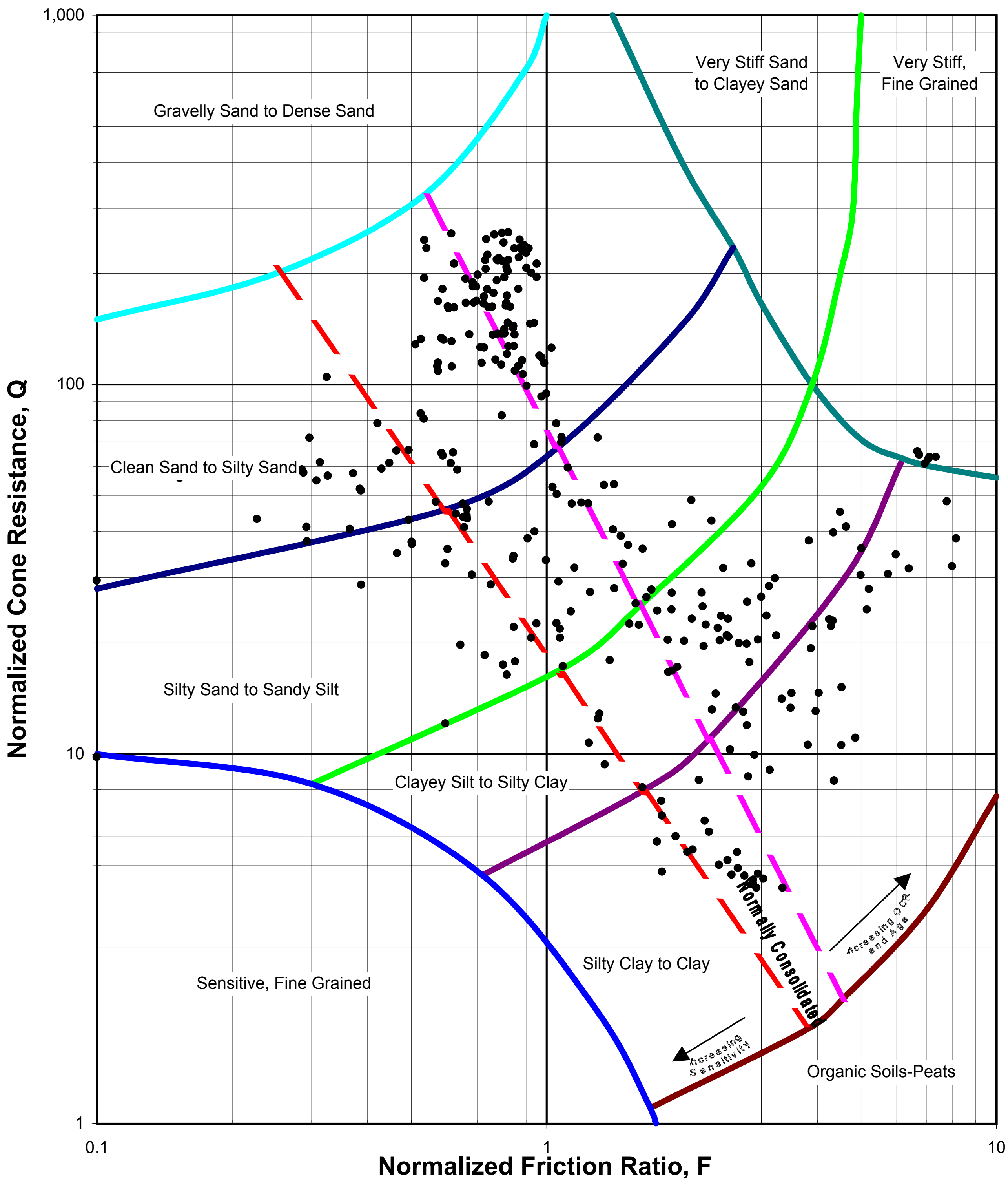
Liquefaction Analysis (CPT-6)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-6.2



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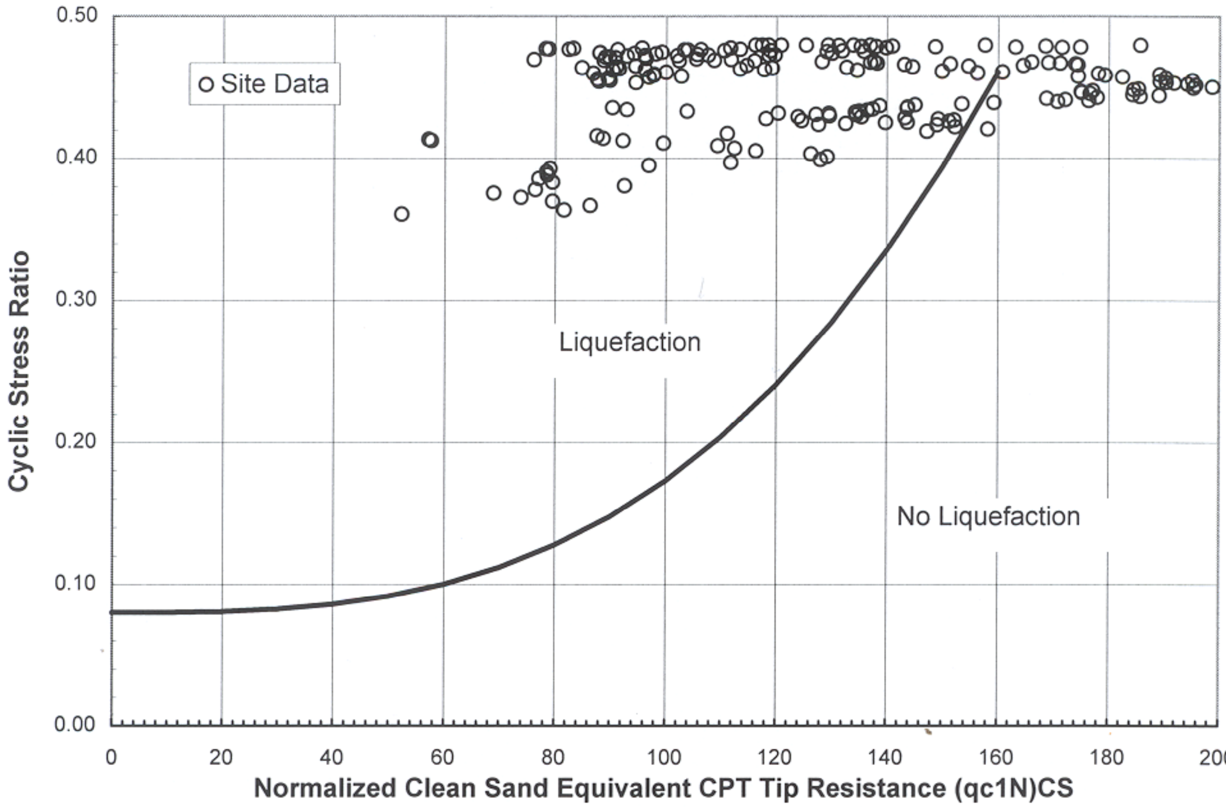
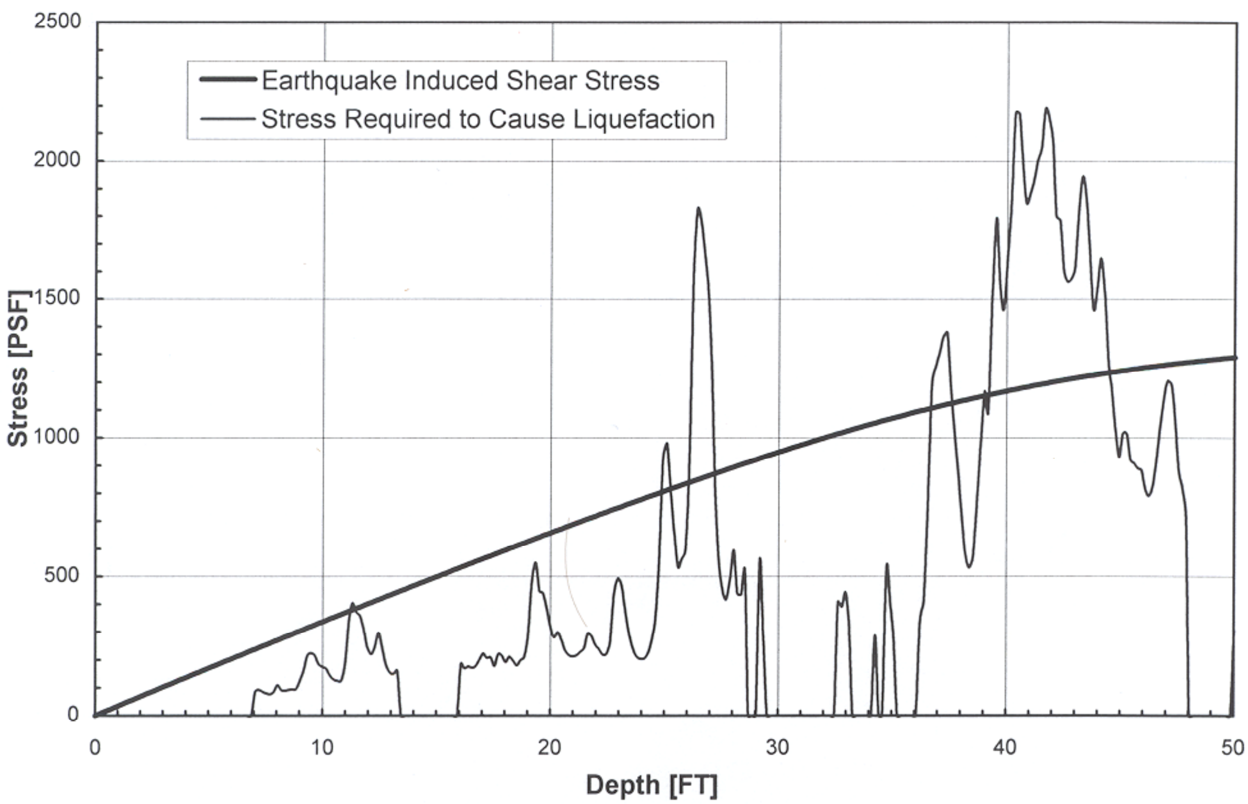
Soil Classification (CPT-6)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-6.3





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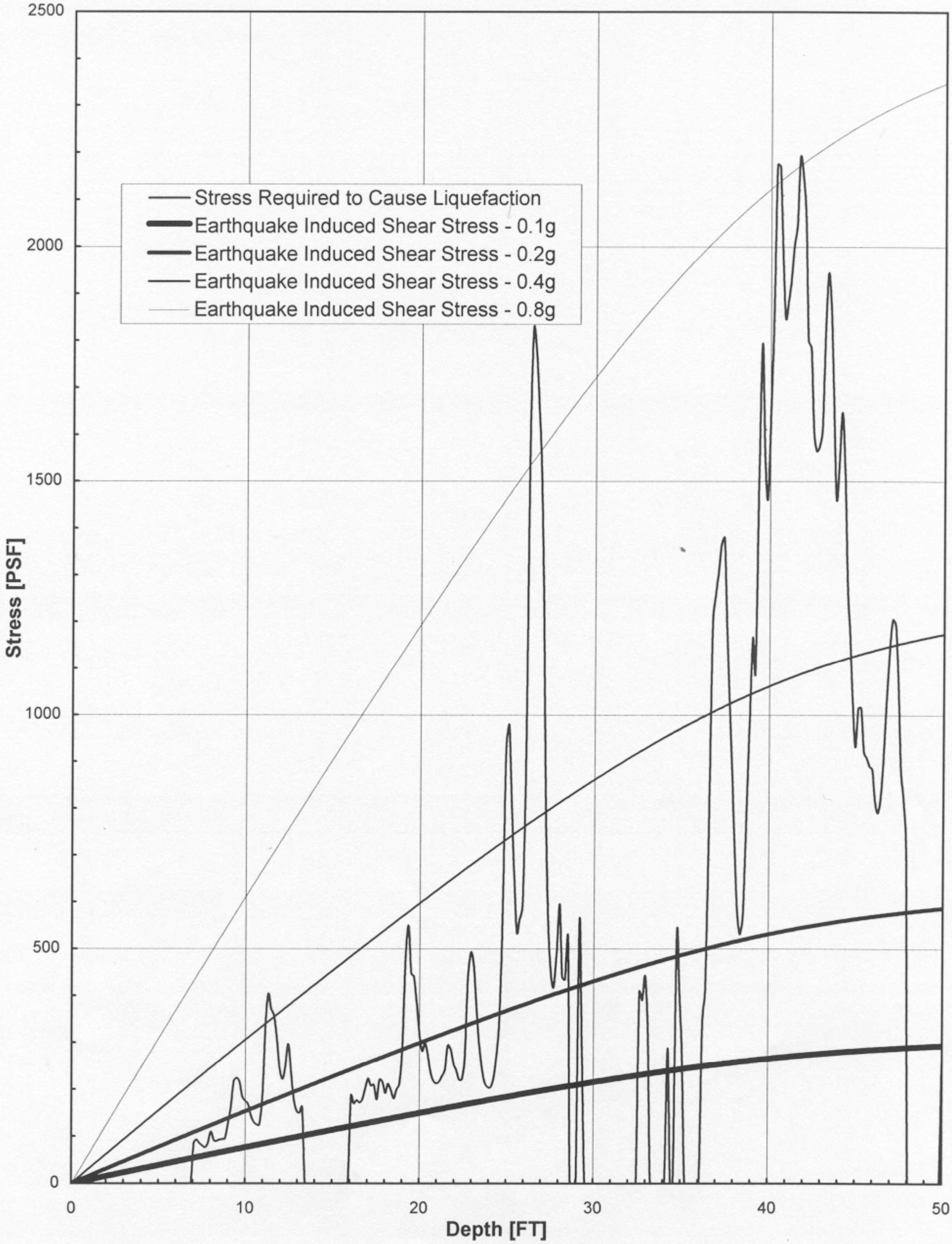
Liquefaction Analysis (CPT-7)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-7.1



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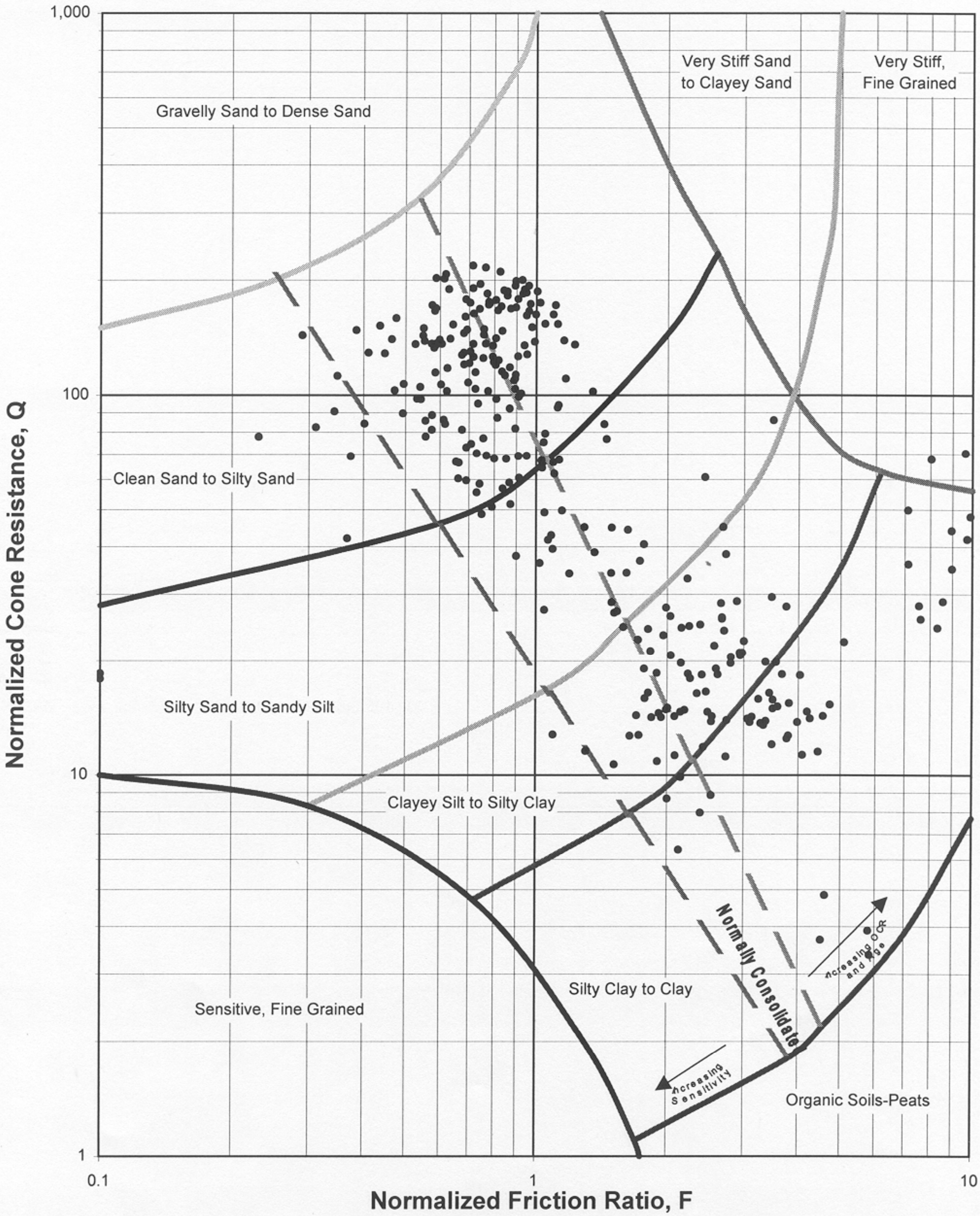
Liquefaction Analysis (CPT-7)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-7.2



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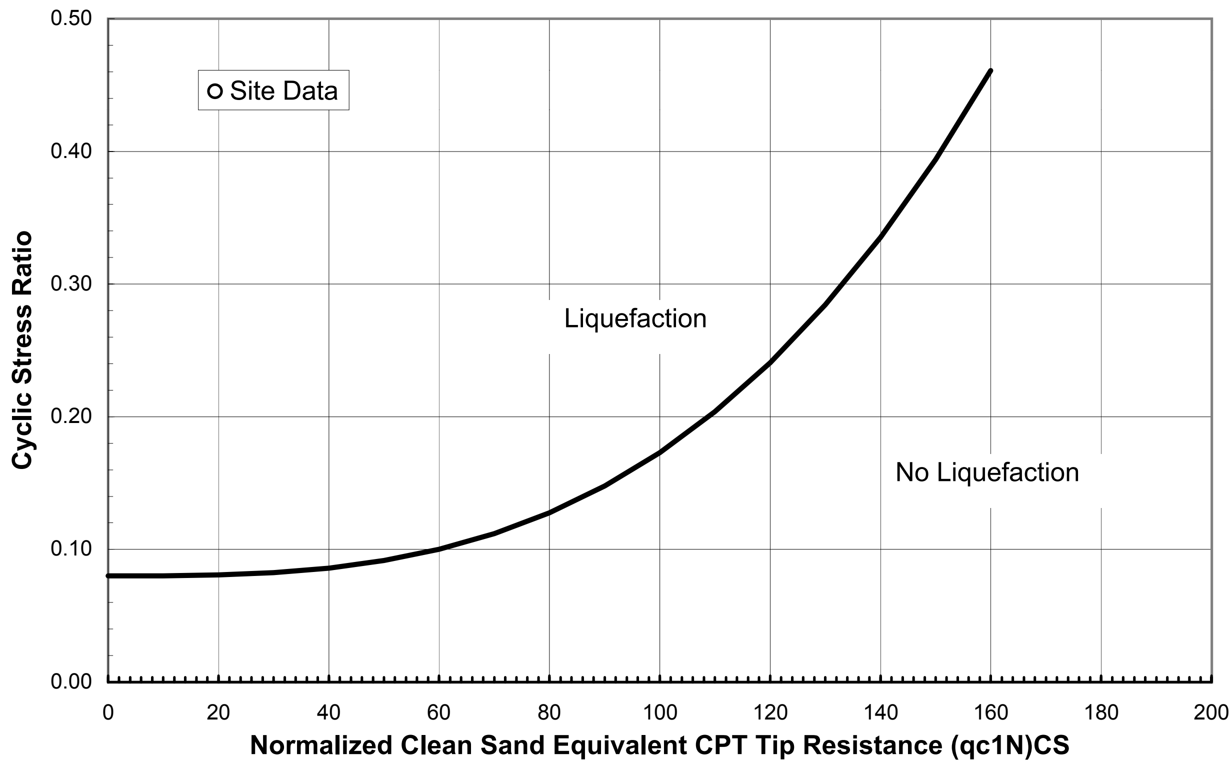
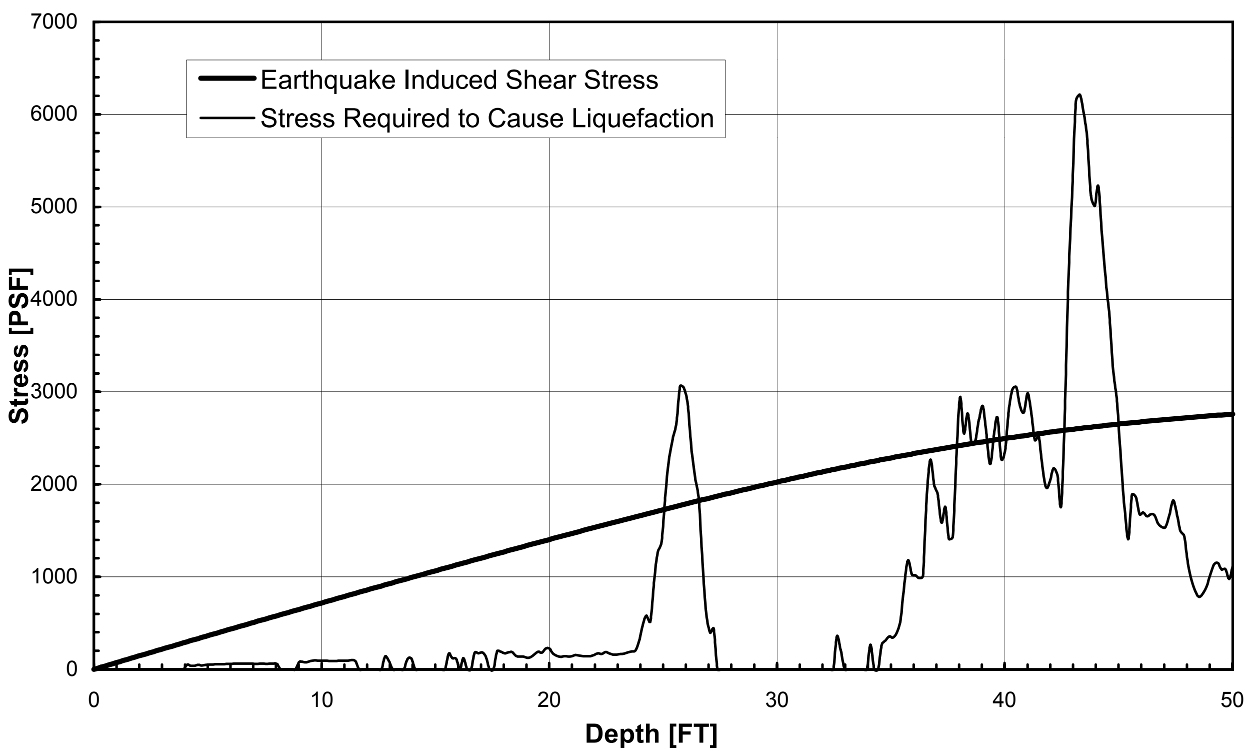
Soil Classification (CPT-7)

Project No. 0673-002-00

Document No. 02-0022

FIGURE E-7.3





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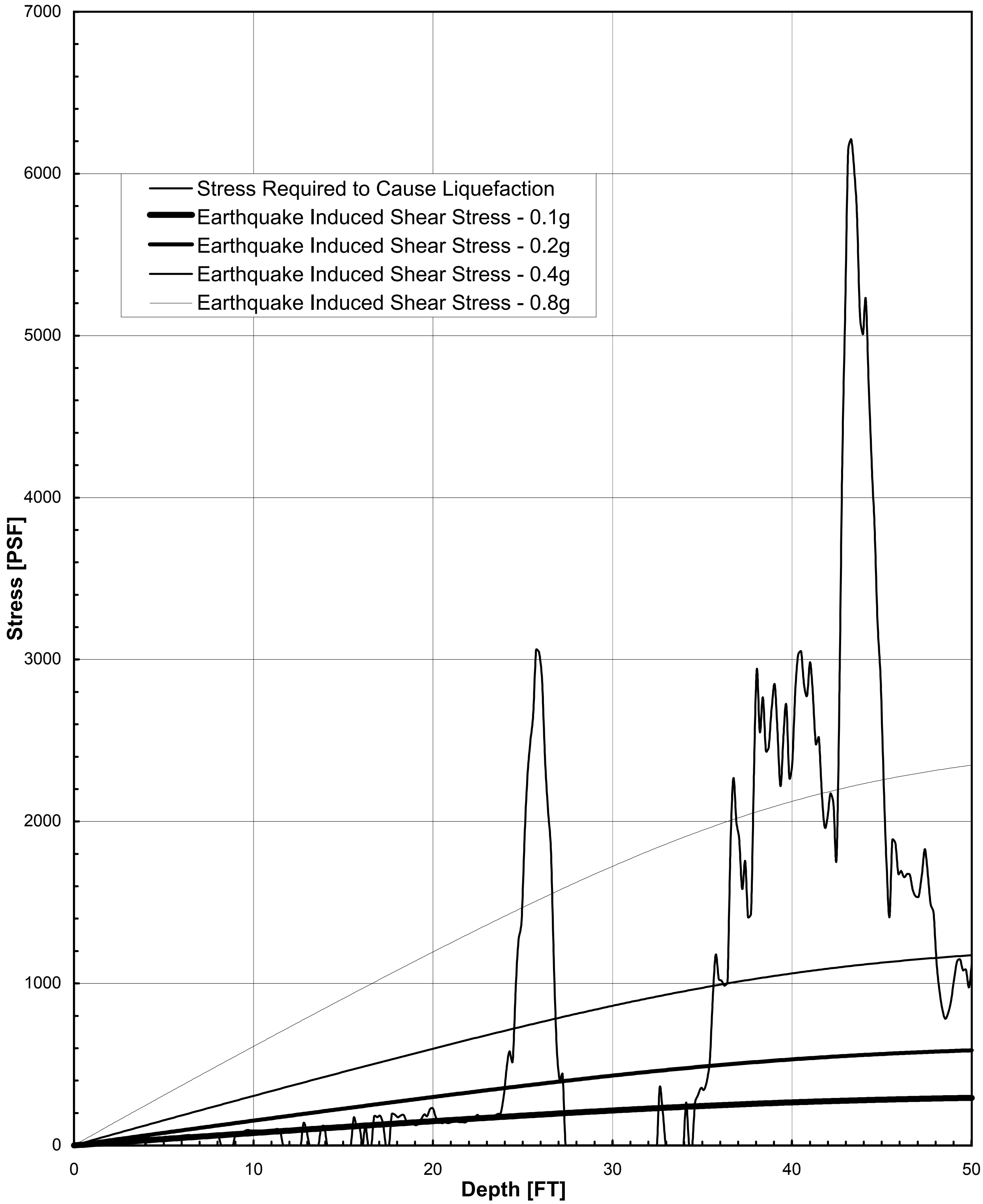
Liquefaction Analysis (CPT-8)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-8.1



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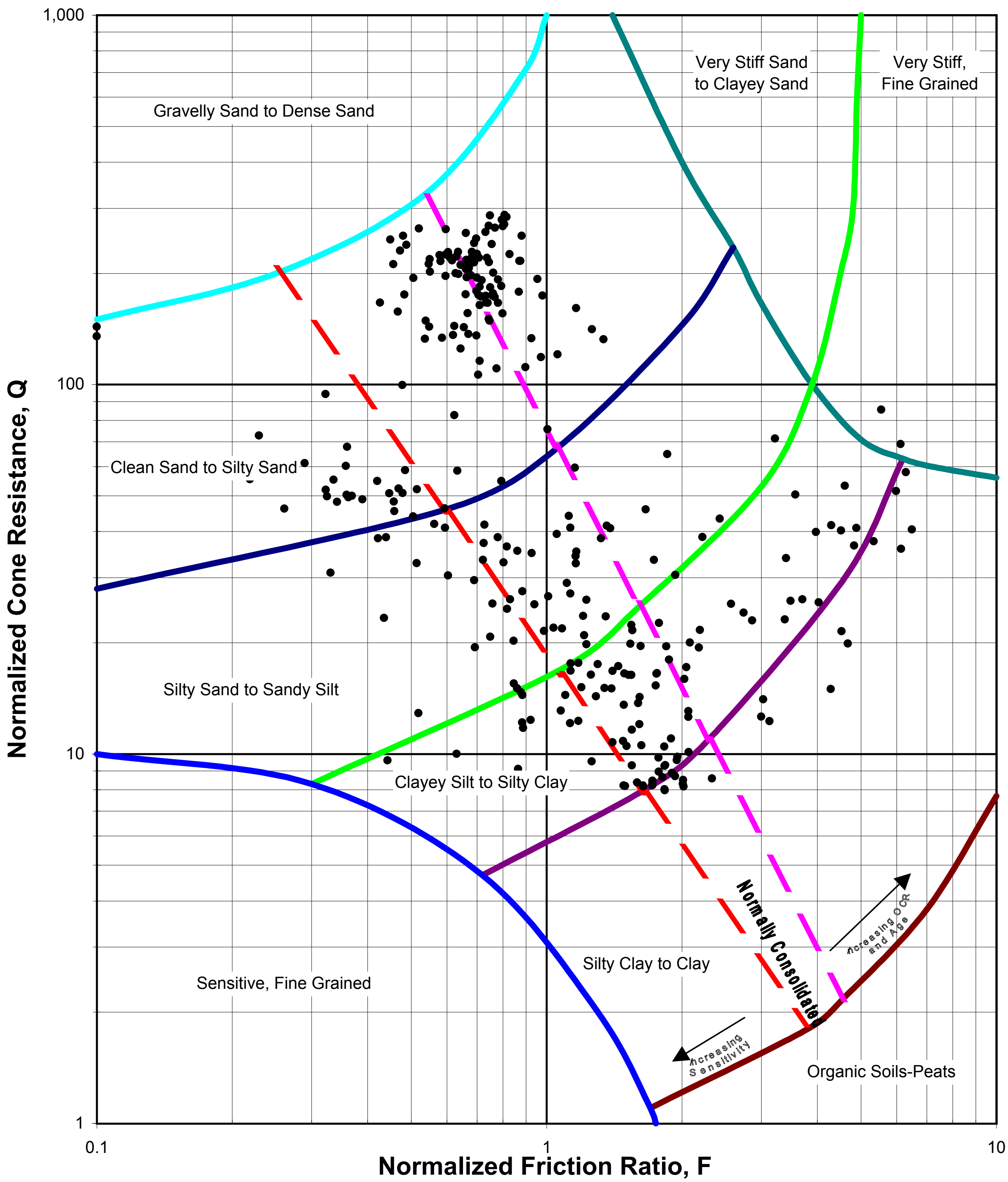
Liquefaction Analysis (CPT-8)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-8.2



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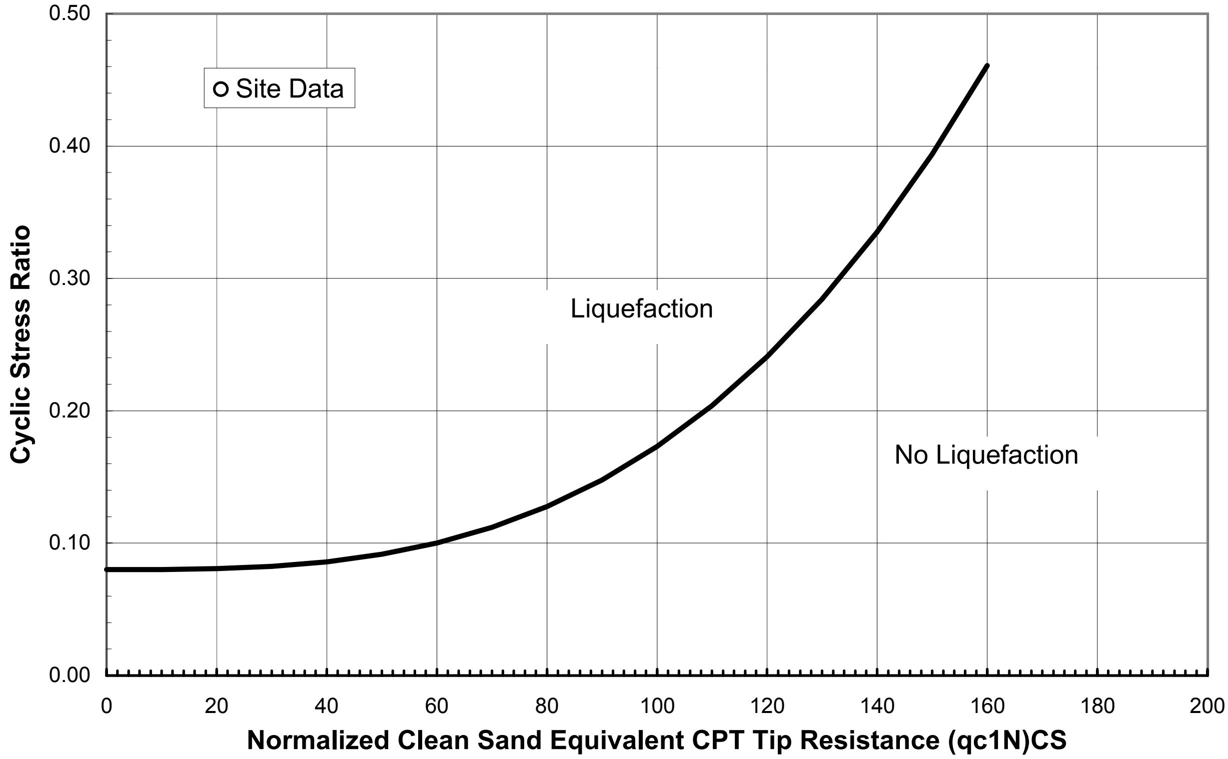
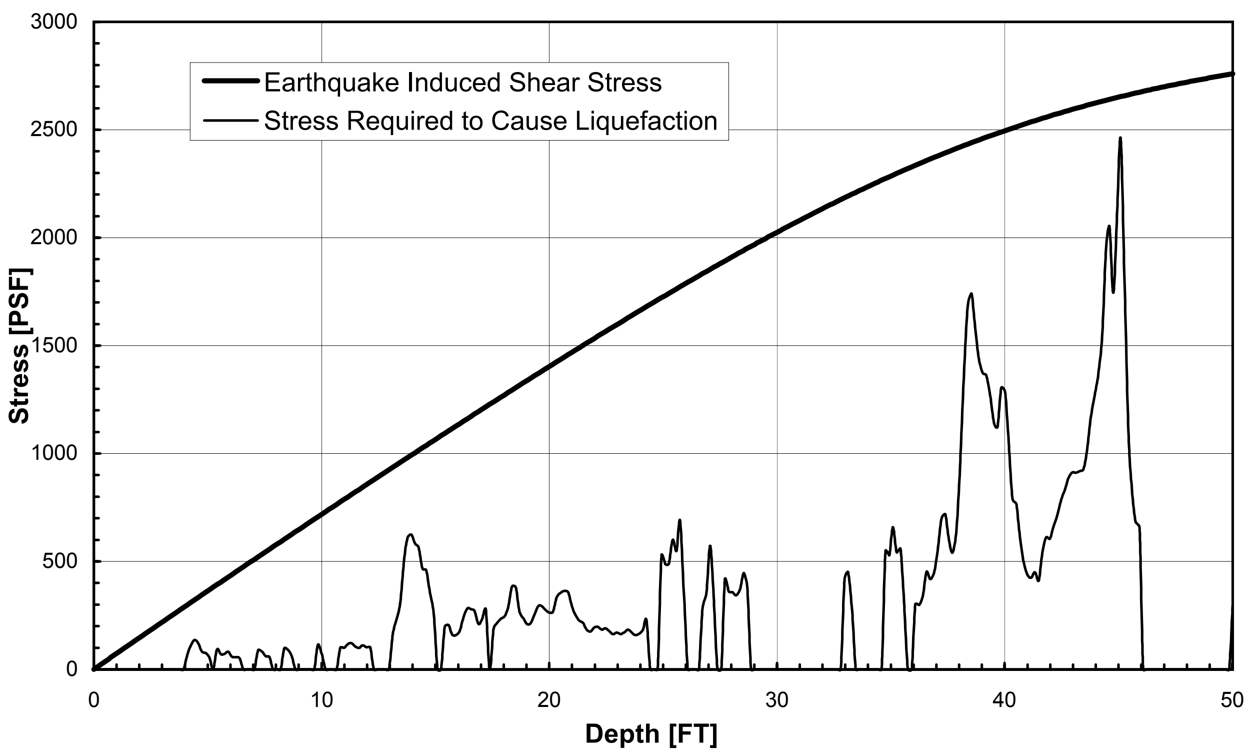
Soil Classification (CPT-8)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-8.3





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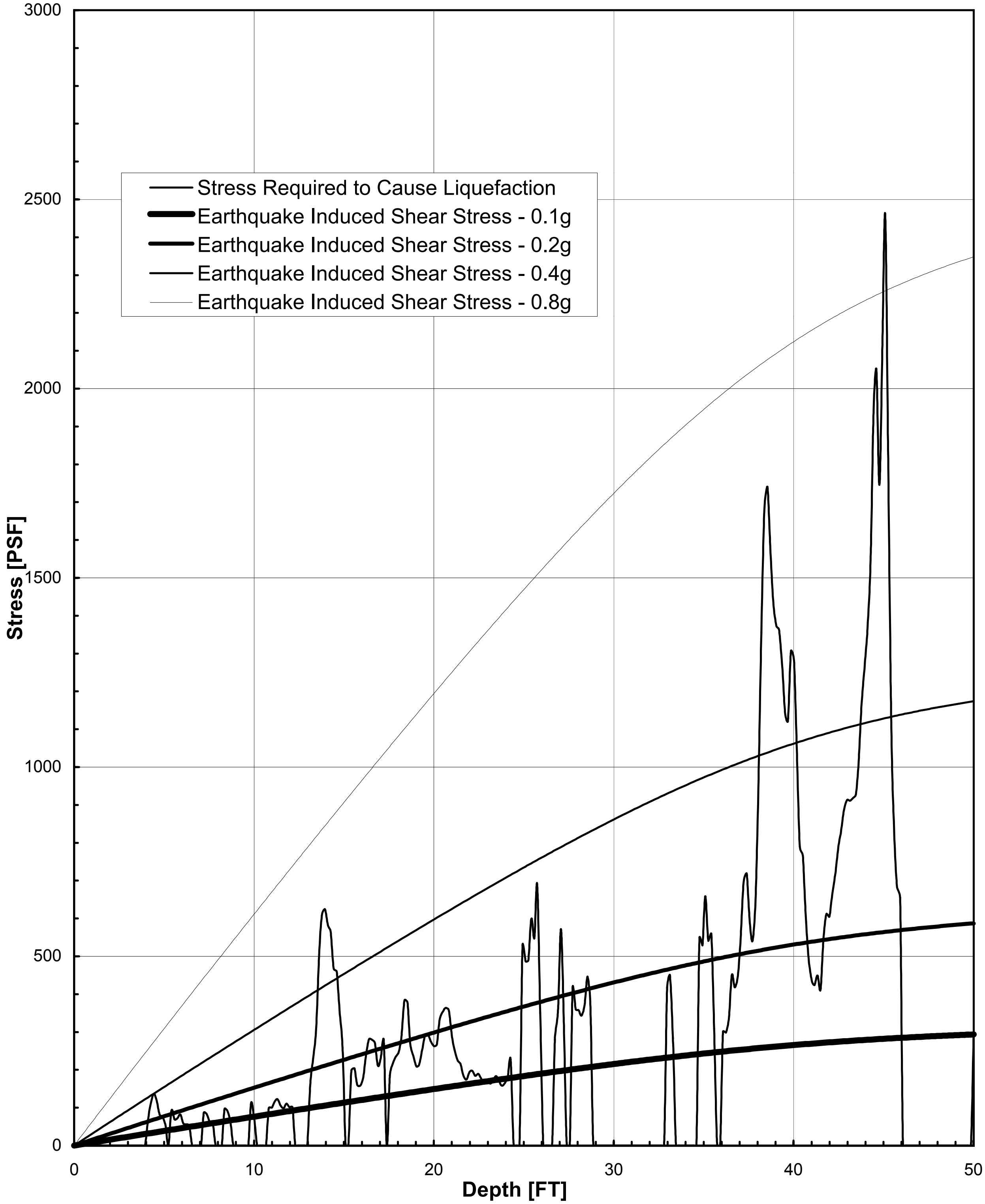
Liquefaction Analysis (CPT-9)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-9.1



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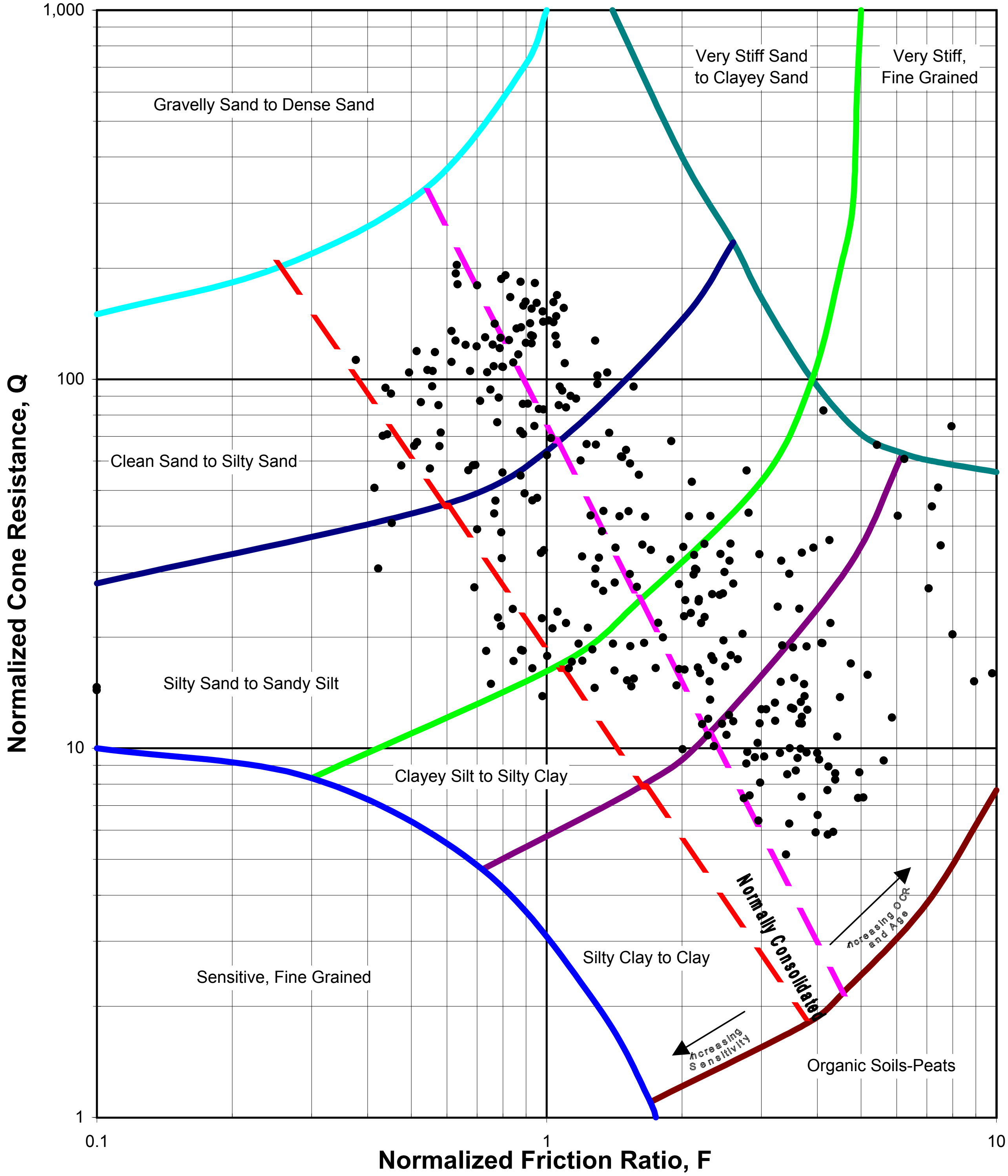
Liquefaction Analysis (CPT-9)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-9.2



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Soil Classification (CPT-9)

Project No. 0673-002-00
Document No. 02-0022
FIGURE E-9.3

APPENDIX F

RESISTIVITY SURVEY

A Wenner four pin resistivity survey was conducted by M.J. Schiff and Associates at the location of the proposed sub-station (approximately 5 feet west of CPT-2). The survey was performed in general accordance with ASTM test method G-57 and IEEE 81. Pin spacings of 2.5, 5, 7.5, 10, 15, 25 and 50 feet were used to develop a resistivity profile with depth. The results are presented below.

M.J. SCHIFF & ASSOCIATES, INC.

Consulting Corrosion Engineers - Since 1959
1308 Monte Vista Avenue, Suite 6
Upland, CA 91786

Phone: (909)931-1360 / Fax: (909)931-1361
E-mail: mjsa@mjs-a.com
http://www.mjs-a.com

December 6, 2001

GEOTECHNICS, INC.
9245 Activity Road # 103
San Diego, CA 92126

Attention: Mr. Matt Fagan

Re: Soil Corrosivity Study
CAL Energy Geothermal Plant-Unit 6
El Centro, California
Your # 0673-002-001, MJS&A #01-1023HQ

INTRODUCTION

Field tests have been completed for the referenced project. Two sets of soil resistivity tests were done at the site. The purpose of these tests was to determine the soil resistivity for electrical grounding design.

TEST PROCEDURES

The electrical resistivity of the soil was measured in two places using the Wenner Four Pin Method per ASTM G-57 and IEEE 81. This procedure gives the average resistivity to a depth equal to the spacing between the pins. Pin spacings of 2.5, 5, 7.5, 10, 15, 25 and 50 feet were used so that variations with depth could be evaluated. Strata resistivities were calculated from resistance data using the Barnes Procedure. Test results are shown on Table 1.

ELECTRICAL GROUNDING

Based on an analysis of the field data, the listed soil resistivity values can be used for electrical grounding design:

For a grounding system using:	Use resistivity value of
shallow, grid	350 ohm-cm
8 or 10-foot ground rods	120 ohm-cm
rods to 25 feet deep	11969 ohm-cm
rods to 50 feet deep	27768 ohm-cm

CLOSURE

Our services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
M.J. SCHIFF & ASSOCIATES, INC.

Adrineh Avakian

Enc: Table 1

Job folders\01-1023hq

Test Date 12/04/01

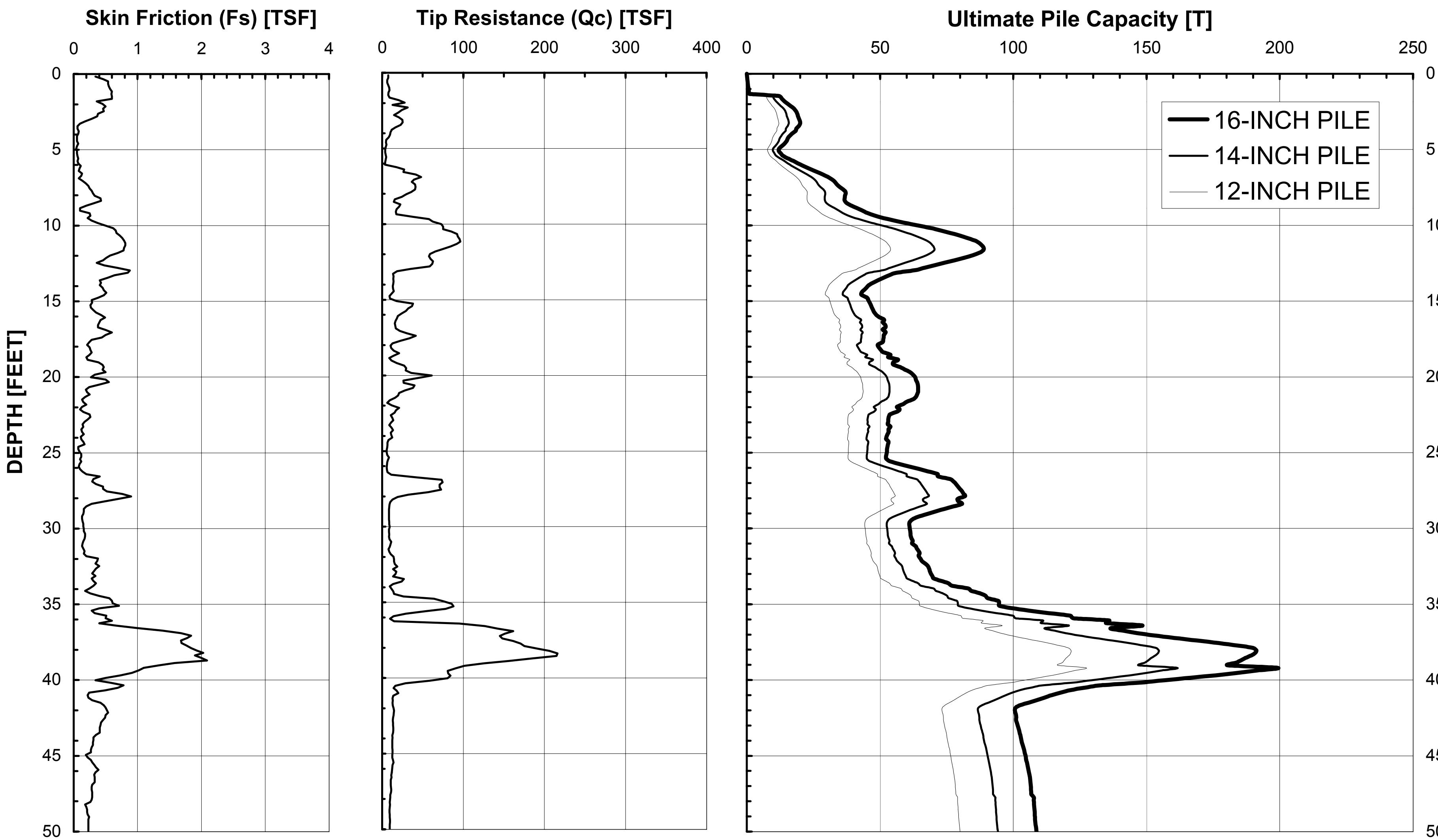
TABLE - 1
SOIL RESISTIVITY - FIELD TESTS
CAL Energy Geothermal Plant-Unit 6, El Centro CA

LOCATION	DEPTH (feet)	MEASURED RESISTANCE (ohms)	AVERAGE RESISTIVITY TO DEPTH (ohm-cm)	STRATUM RESISTIVITY (ohm-cm)
	2.5	0.70	350	
				140
	5.0	0.20	200	
				100
	7.5	0.10	150	
				50
	10.0	0.05	100	
				50
	15.0	0.03	75	
				-49
	25.0	1.90	9096	
				19709
	50.0	1.30	12448	
	2.5	0.70	350	
				183
	5.0	0.24	240	
				120
	7.5	0.12	180	
				60
	10.0	0.06	120	
				60
	15.0	0.03	90	
				-58
	25.0	2.50	11969	
				-86773
	50.0	2.90	27768	

APPENDIX G

PILE LOAD ANALYSIS

To aid in developing the allowable pile capacities presented in this report, load analysis was conducted using the CPT data collected during the subsurface exploration. The ultimate pile capacities were estimated in general accordance with the LCPC Method (Bustamante and Gianeselli, 1982). The analysis assumed that driven, precast, square concrete piles would be used. Diameters of 12, 14 and 16 inches were used, and no cut-off was included for the upper limits of pile capacity. The results of our analysis are presented in Figures G-1 through G-9. The lower bounds of these curves are presented in Figures G-10 through G-12 for 12, 14, and 16 inch square precast concrete piles, respectively. Note that the ultimate pile capacities presented in these figures do not include a safety factor.



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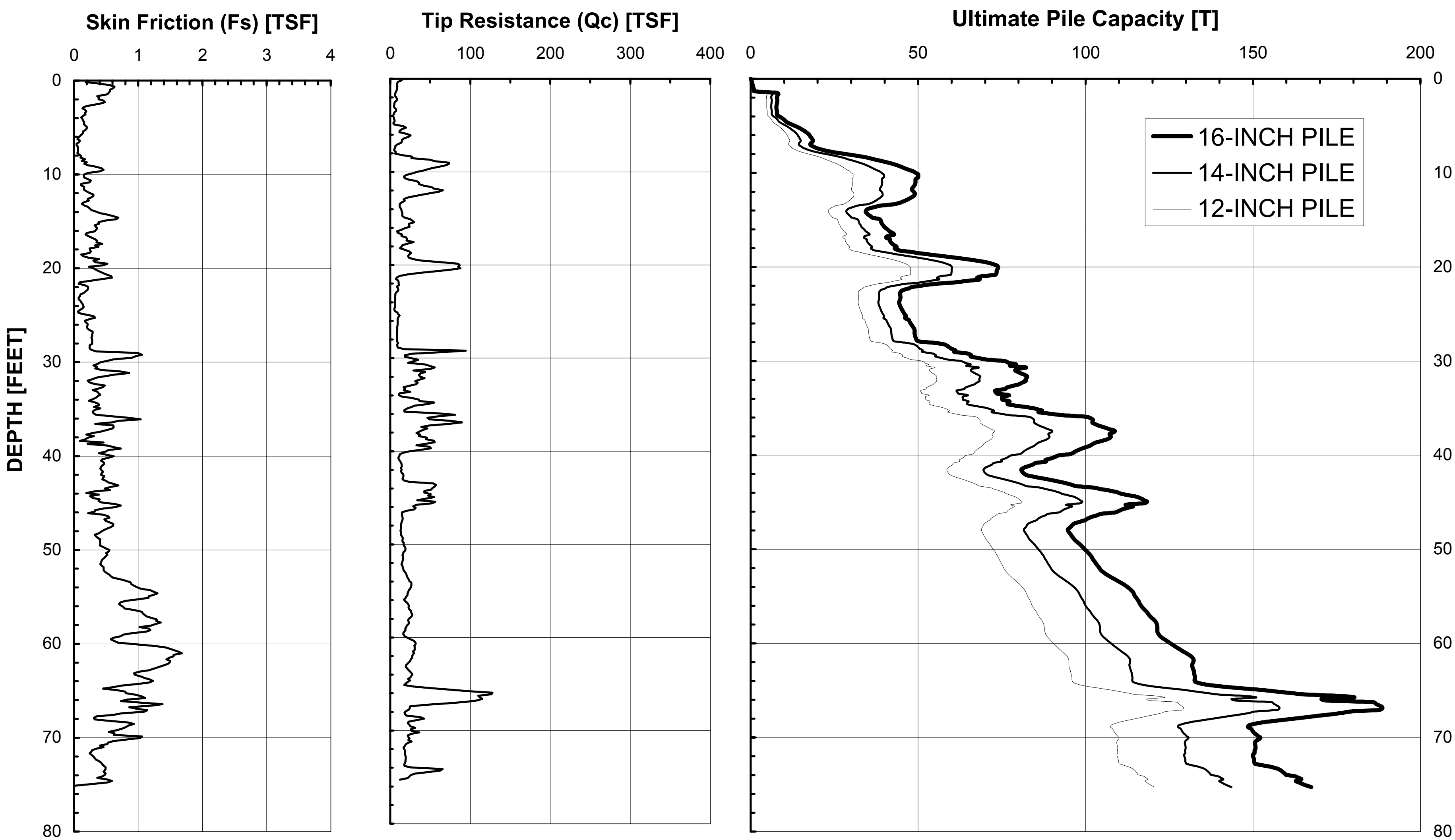
PILE CAPACITY CHART (CPT-1)

Project No. 0673-002-00
Document No. 02-0022
FIGURE G-1



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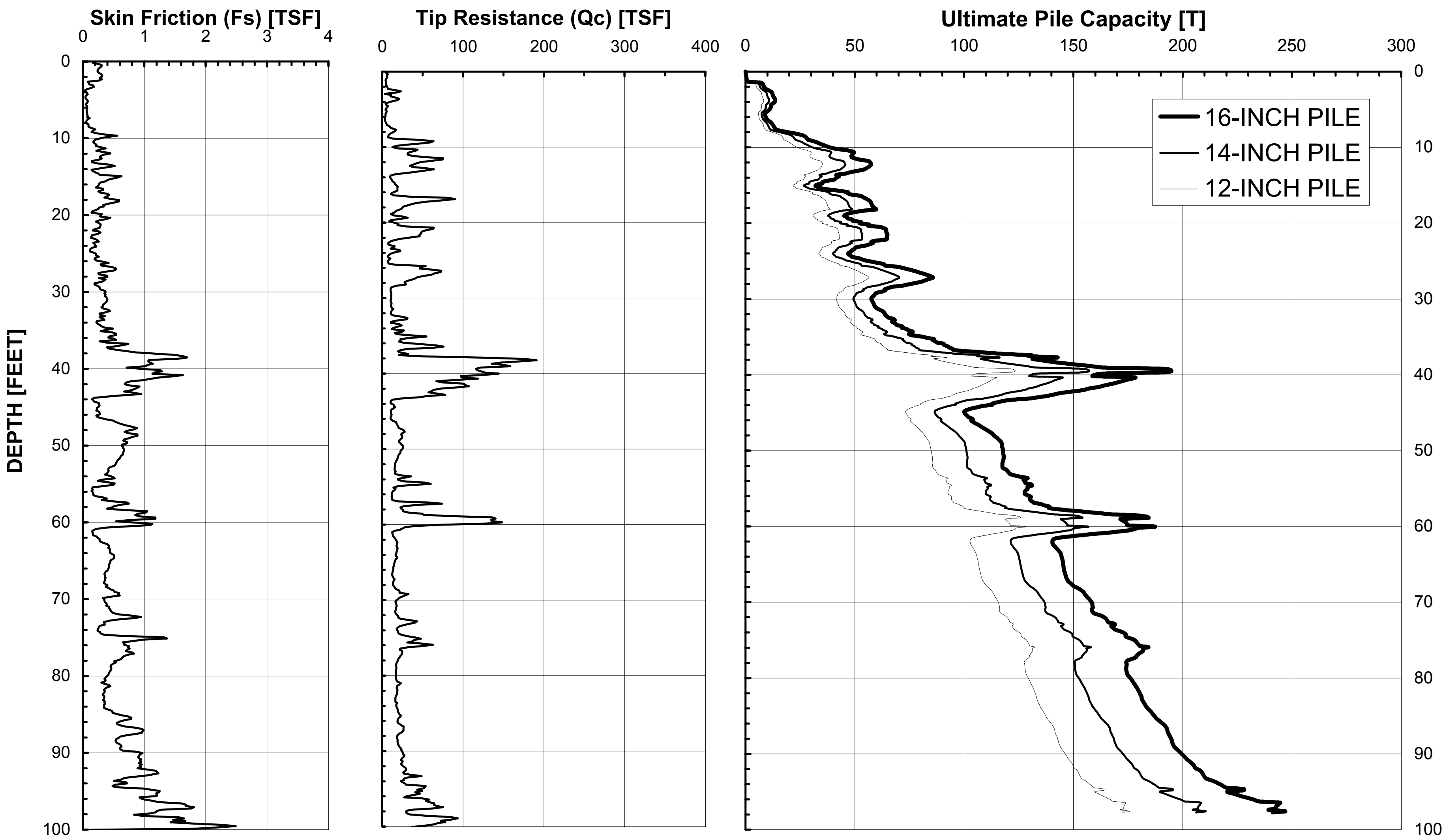
PILE CAPACITY CHART (CPT-2)

Project No. 0673-002-00
Document No. 02-0022
FIGURE G-2



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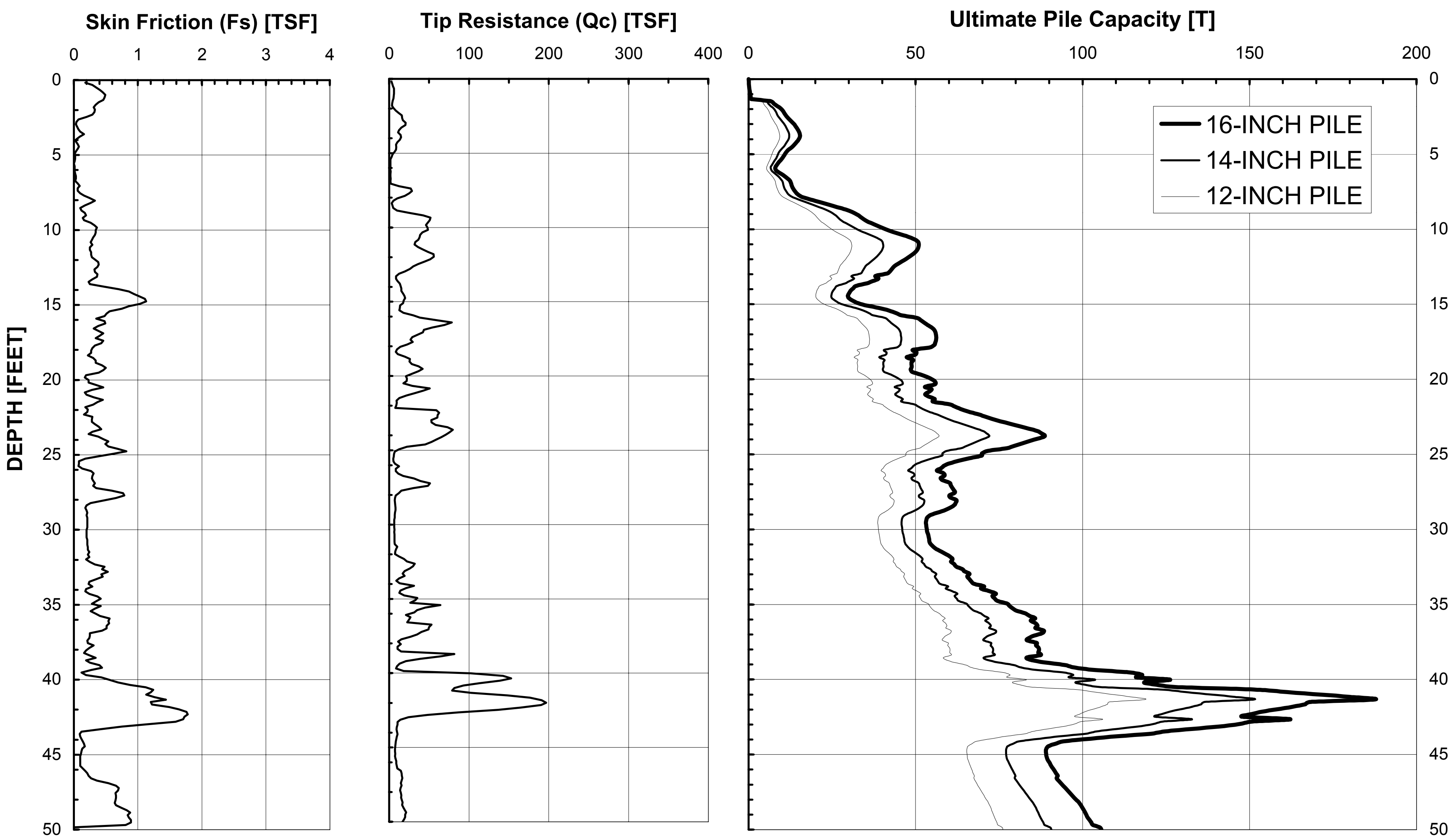
PILE CAPACITY CHART (CPT-3)

Project No. 0673-002-00
Document No. 02-0022
FIGURE G-3



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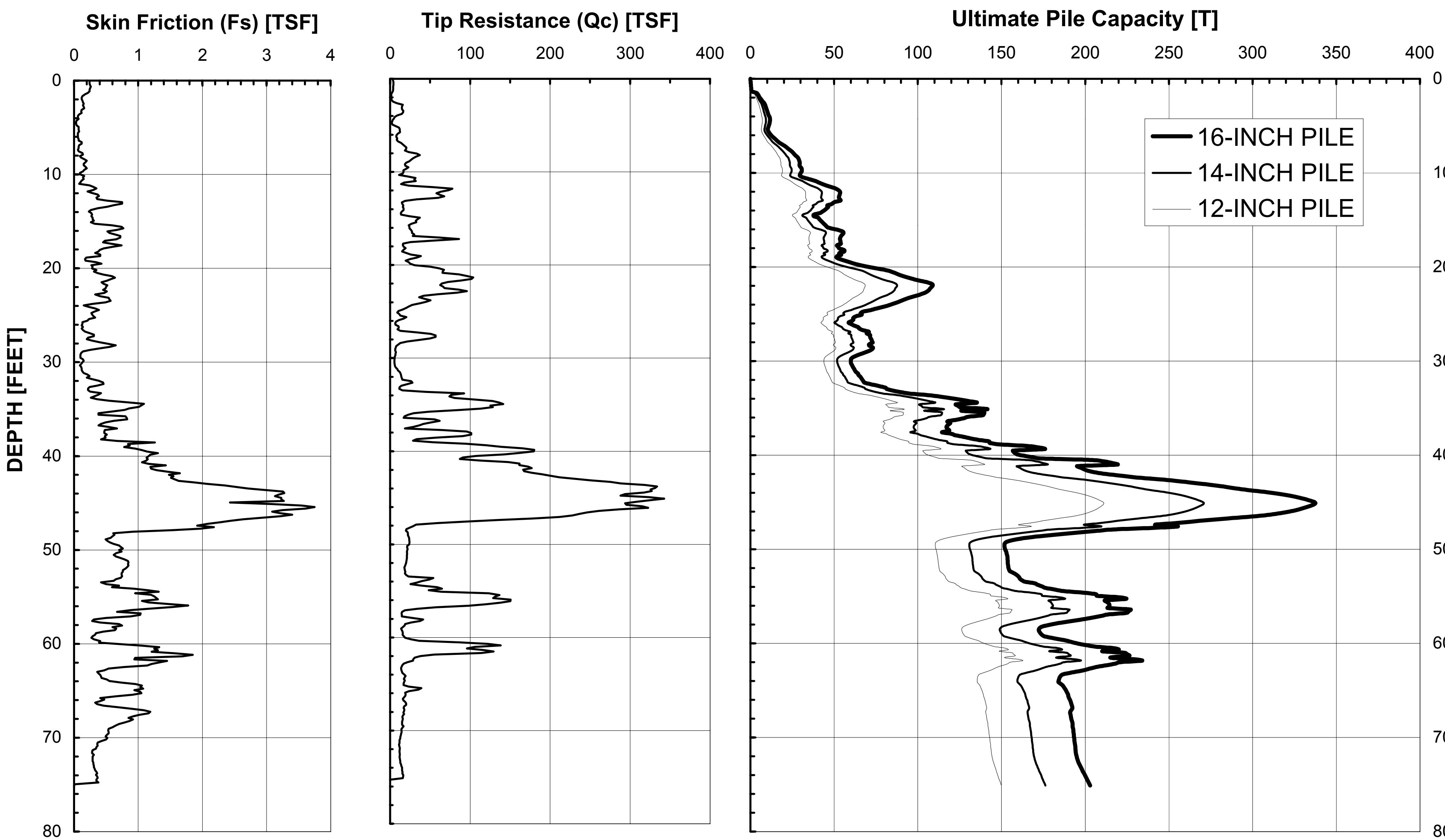
PILE CAPACITY CHART (CPT-4)

Project No. 0673-002-00
Document No. 02-0022
FIGURE G-4



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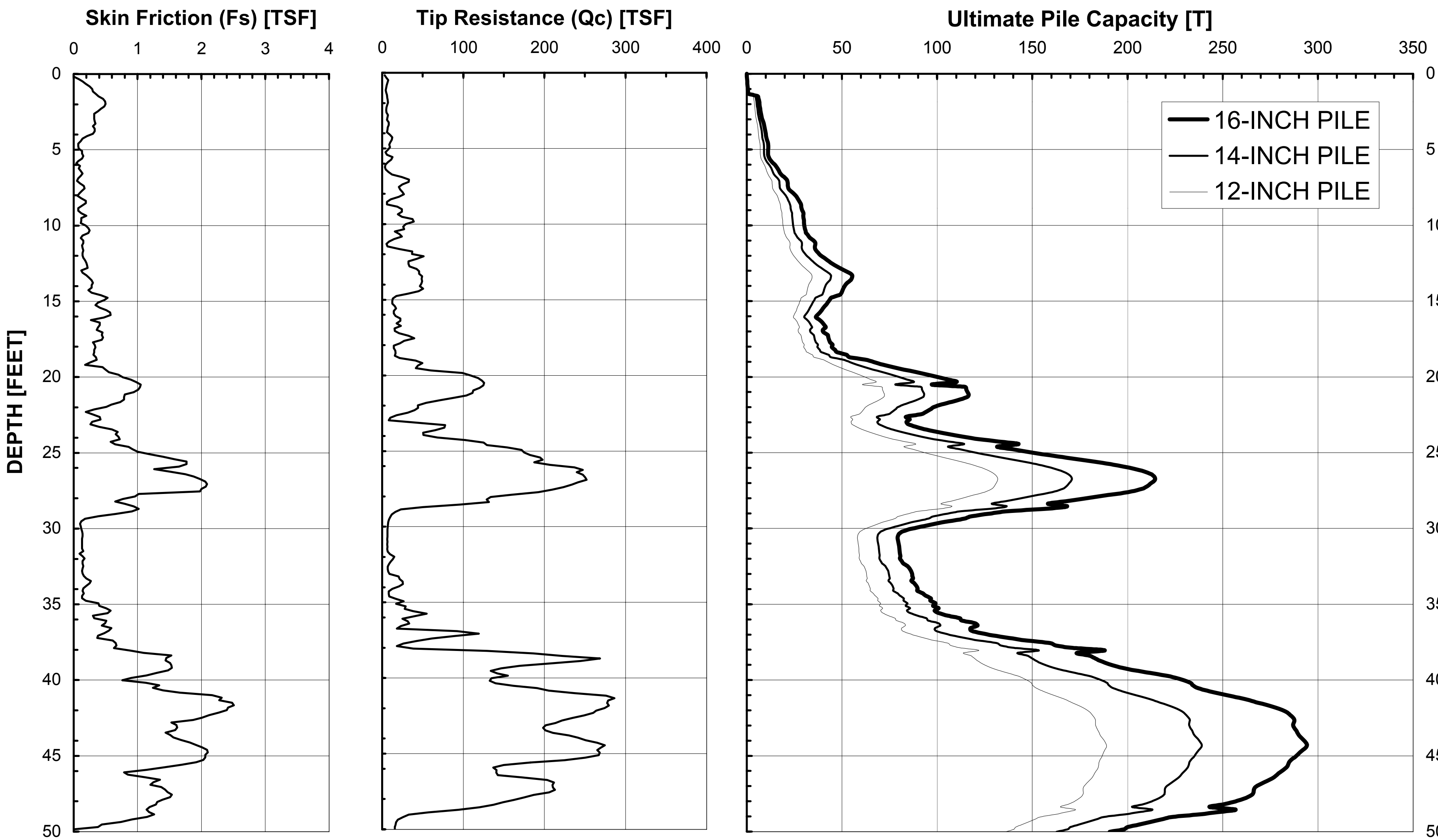
PILE CAPACITY CHART (CPT-5)

Project No. 0673-002-00
Document No. 02-0022
FIGURE G-5



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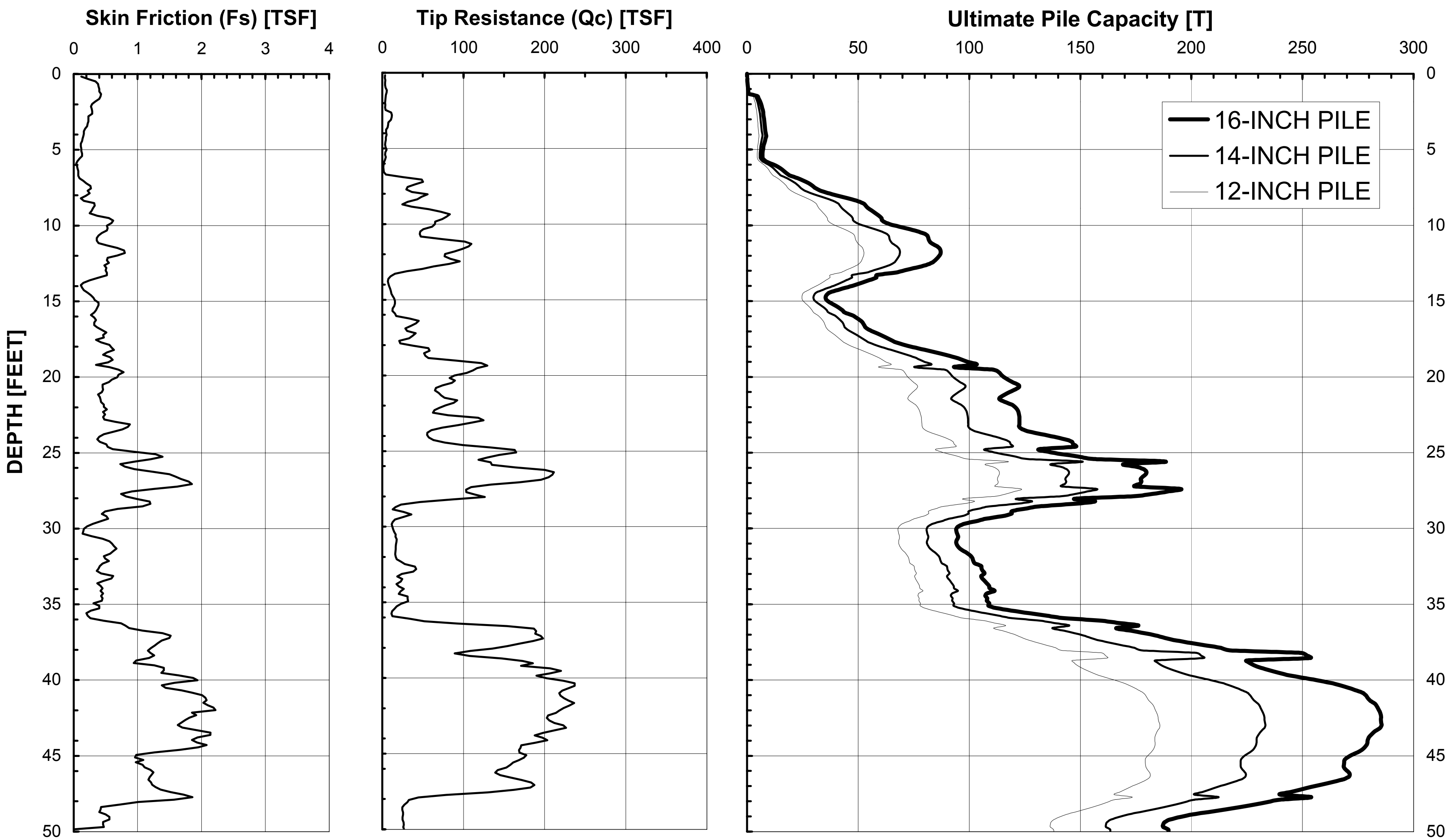
PILE CAPACITY CHART (CPT-6)

Project No. 0673-002-00
Document No. 02-0022
FIGURE G-6



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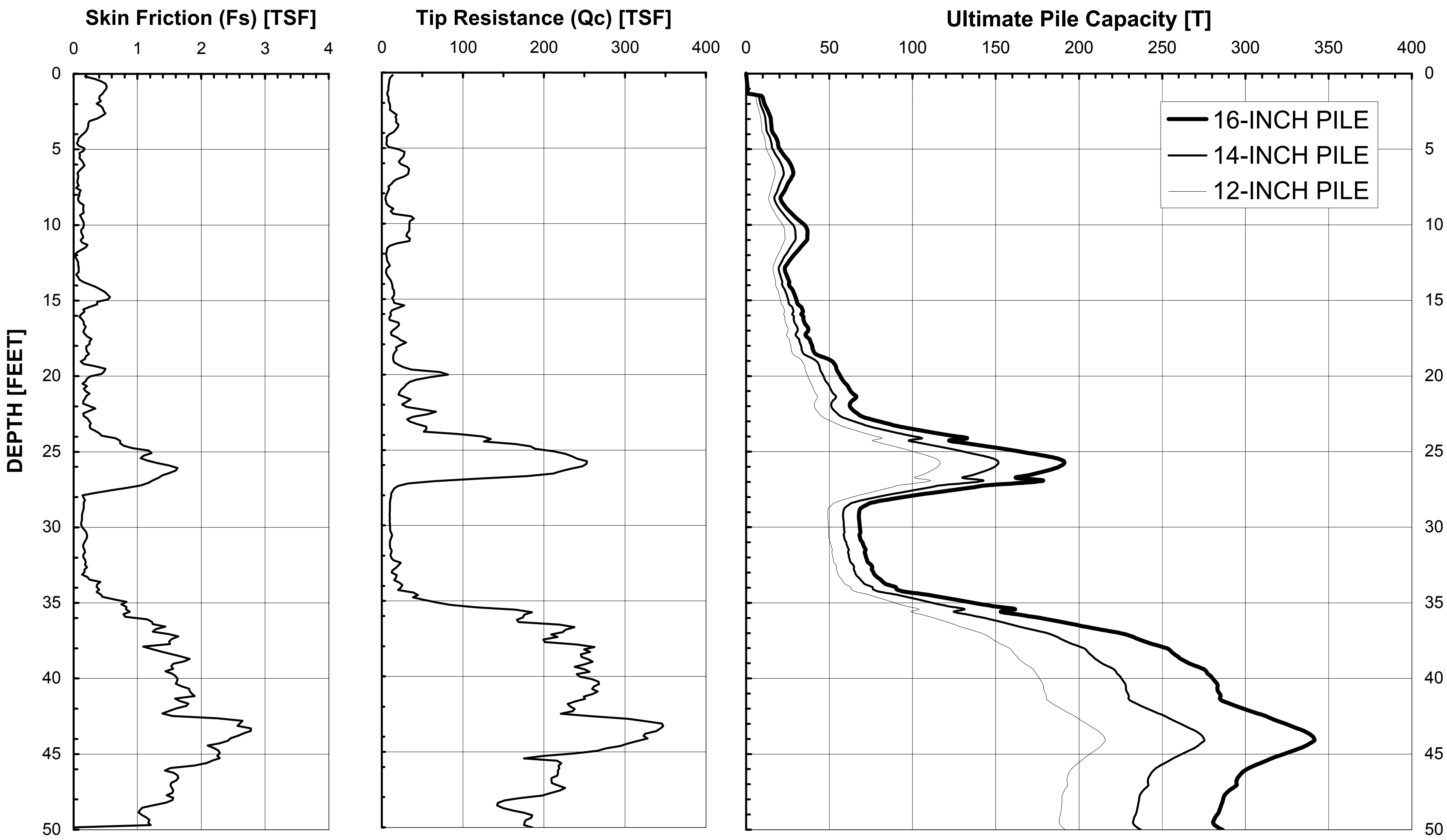
PILE CAPACITY CHART (CPT-7)

Project No. 0673-002-00
Document No. 02-0022
FIGURE G-7



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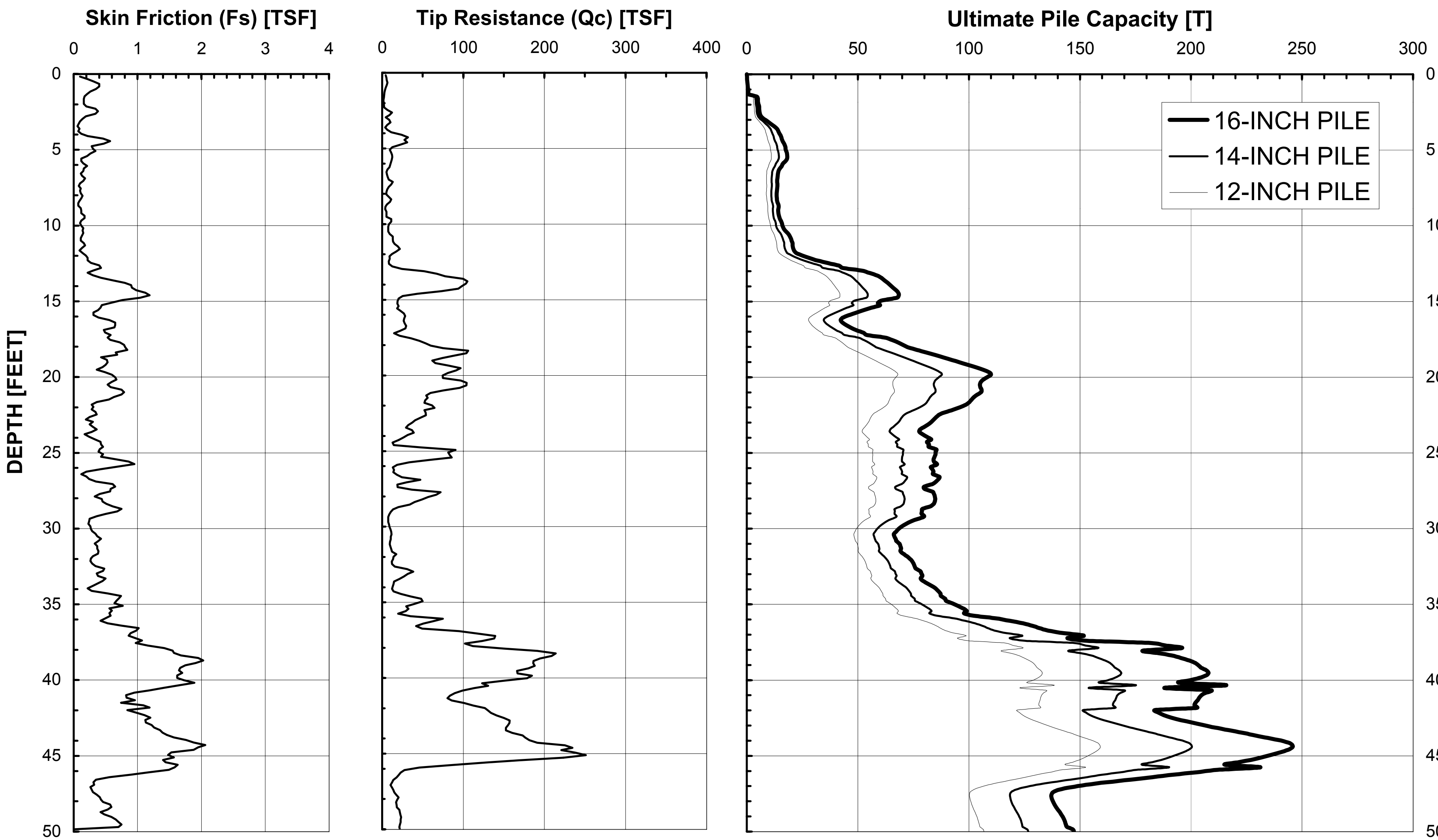
PILE CAPACITY CHART (CPT-8)

Project No. 0673-002-00
Document No. 02-0022
FIGURE G-8



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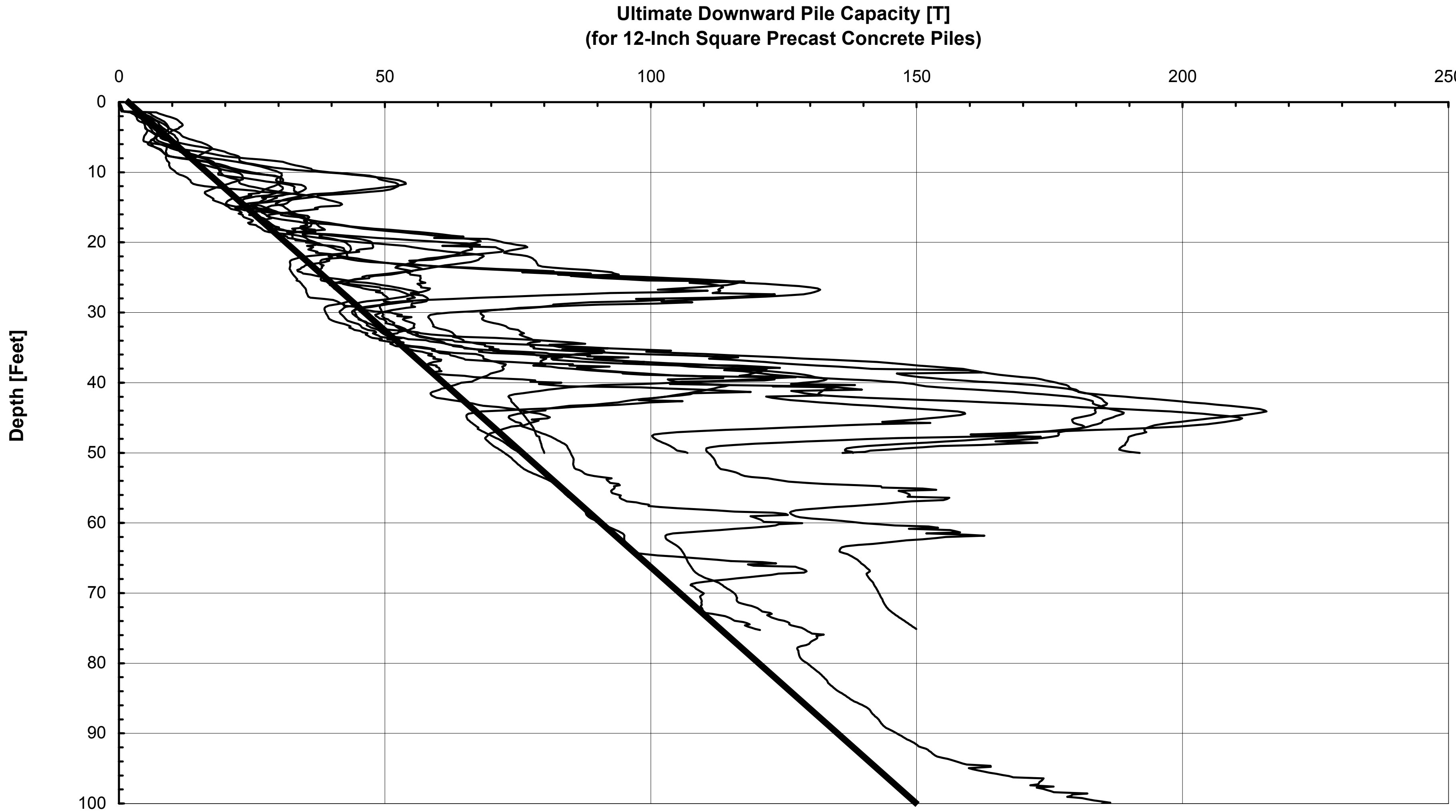
PILE CAPACITY CHART (CPT-9)

Project No. 0673-002-00
Document No. 02-0022
FIGURE G-9



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PILE CAPACITY SUMMARY

Project No. 0673-002-00
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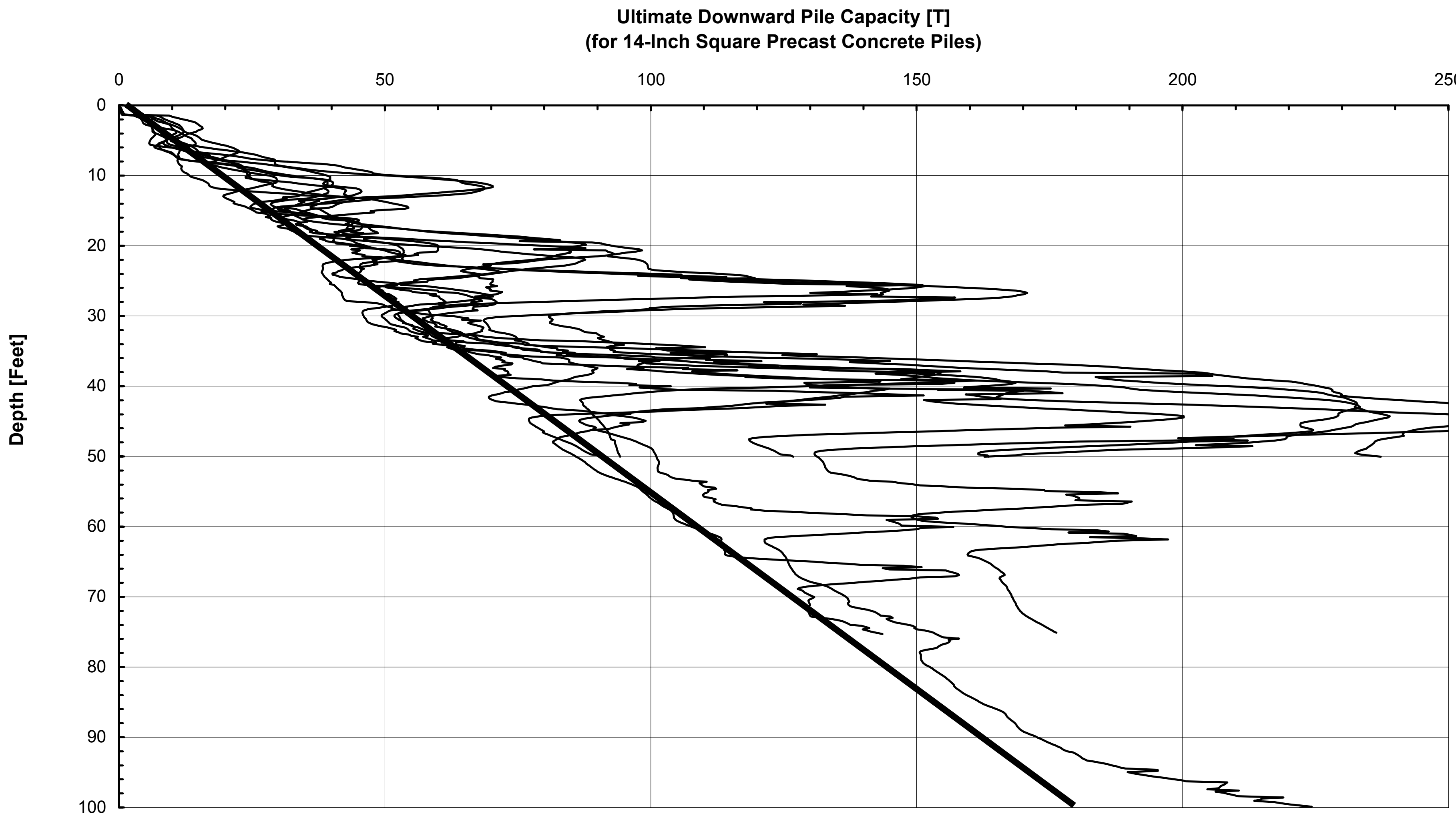
FIGURE G-10



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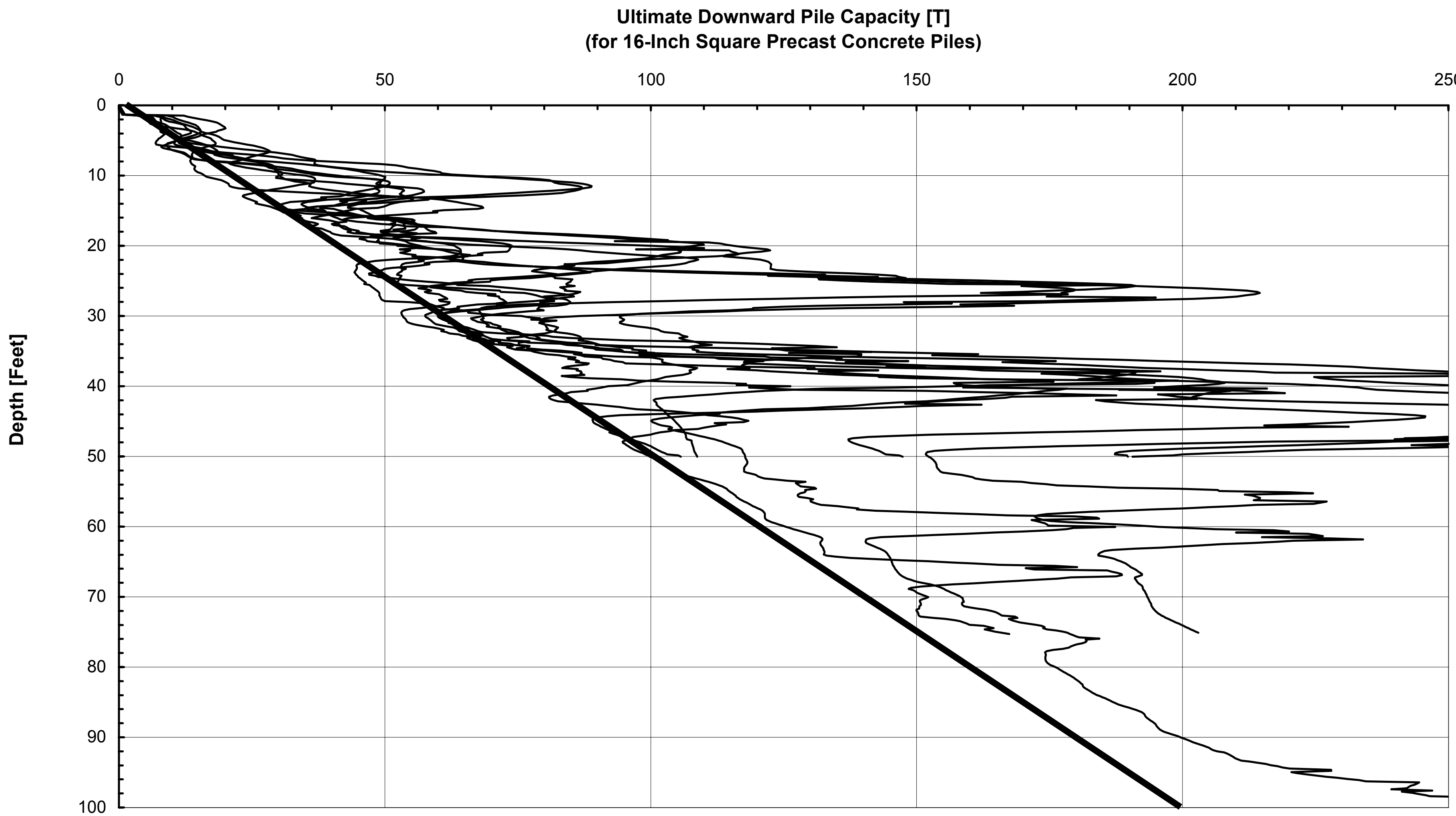
PILE CAPACITY SUMMARY

Project No. 0673-002-00
Document No. 02-0022
FIGURE G-11



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PILE CAPACITY SUMMARY

Project No. 0673-002-00
Document No. 02-0022
FIGURE G-12